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***PAVEMENT AND
SUBGRADE DISTRESS –
REMEDIAL STRATEGIES
FOR CONSTRUCTION AND
MAINTENANCE (I-15
Mileposts 200-217)***

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UDOT RESEARCH & DEVELOPMENT REPORT ABSTRACT

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16. Abstract Sections of Interstate 15 within a 17 miles length of roadway from about mileposts 217 to 200 south of Nephi, Utah have been experiencing considerable distress since construction. Maintenance costs have been significant and it appears that distress may not simply be due to an inadequate pavement section. The problems associated with bumps, cracks and edge failures are likely associated with troubles in the subgrade soils along the alignment. Potential causes could include collapsible soil, expansive soil, compressible soil, poorly compacted fill and poor drainage. The objectives of this research study are to determine the causes for the problems and potential solutions prior to design and reconstruction of the area in question. Based on surficial geology and borehole data, zones were identified where collapsible soils were likely the culprit. Because the zone of collapsible soil extends to depths of up to 20 ft below the ground surface, deep dynamic compaction was recommended over excavation and replacement as a treatment method in these zones. Distress related to expansive soils exists throughout the study area, but significant damage concentrations are located in a cut section between mileposts 208 and 207 along I-15. This area is long enough to propose treatments for the area, in order to improve ride quality throughout the cut section. This study recommends a combination of methods as illustrated in Figure ES-3 to improve the odds of success. Because of the potential for differential settlement on the roadway, asphalt pavement should be used in reconstructing the roadway in the study area. A lack of adequate surface drainage is another critical factor leading to problems with both collapsible and expansive subgrade soils in this area.					
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(I-15 Mileposts 200-217)**

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EXECUTIVE SUMMARY

The objectives of this investigation were to: (1) identify locations of distressed highway sections, (2) evaluate likely causes of distress, and (3) to recommend potential treatment methods to reduce the potential for future distress.

The study identified distress locations along I-15 from milepost 200 to 217. These distress features included bumps, depressions, cracks and fragmented pavement at the edge of the roadway (Edge Failures). This investigation indicated that about half of the distress features were concentrated between milepost 207 and 210. Distress features were nearly evenly distributed between cuts or fills; however, about 30% of the distress features were located with 100 ft of cut/fill transitions. In addition, between 30 to 40% of the distress features were located with 100 ft of a culvert or drainage path.

In terms of geology, 30% of the distress features were located within alluvial soils which would likely be collapsible and 12% within volcanic tuffs, mudstones and shales which would have a tendency to be expansive when weathered. A total of nine exploratory borings were drilled during the course of this investigation. In addition, logs from 28 borings previously drilled at locations of overpass structures within the study area were collected. Undisturbed samples obtained during this study were tested and expansion/contraction problems were identified in the soil (weathered rock) from the cut sections of the highway. In addition, collapse characteristics were identified in the silty sands, silts and silty clays located in the alluvial fan zones.

Collapsible Soil Treatment Recommendations

Based on the surficial geology and the borehole data, zones were identified where collapsible soils were likely the problem. Figure ES-1 provides a close-up view which shows the zones relative to the borehole locations and distress points. The combined length of the four zones is 0.735 miles. The first zone starts, for both south and northbound lanes, at mile 209.728 and ends at mile 209.412 for a length of 1670 feet. The second zone, starts at mile 209.308 and ends at mile 209.245 for a length of 332 feet. The third zone starts at mile 209.22 and ends at mile 209.153 for a length of 352 feet. The fourth zone, for both the south and northbound lanes, starts at mile 209.108 and ends at mile 208.814 for a length of 1550 feet. These boundaries will likely need to be refined during construction as more information on the exact boundaries is uncovered. The present boundaries are somewhat conservative and extend through the fan material and into the cut sections in some cases.

Because the zone of collapsible soil extended to depths of up to 20 ft below the ground surface, deep dynamic compaction was recommended over excavation and replacement as a treatment method in these zones. Deep dynamic compaction has been employed with success in treating collapsible soil problems below interstate highways in New Mexico, Wyoming and Montana. This in-situ treatment method is one of the most economical soil improvement methods available (approximately \$1 to \$1.20 square foot of surface area) Recommendations for drop weight and height, energy per surface area, drop spacing and number of drops are presented in the body of the report. *After dynamic compaction every effort should be made to minimize water infiltration due to ponding at culverts and within other surface drainage features.*

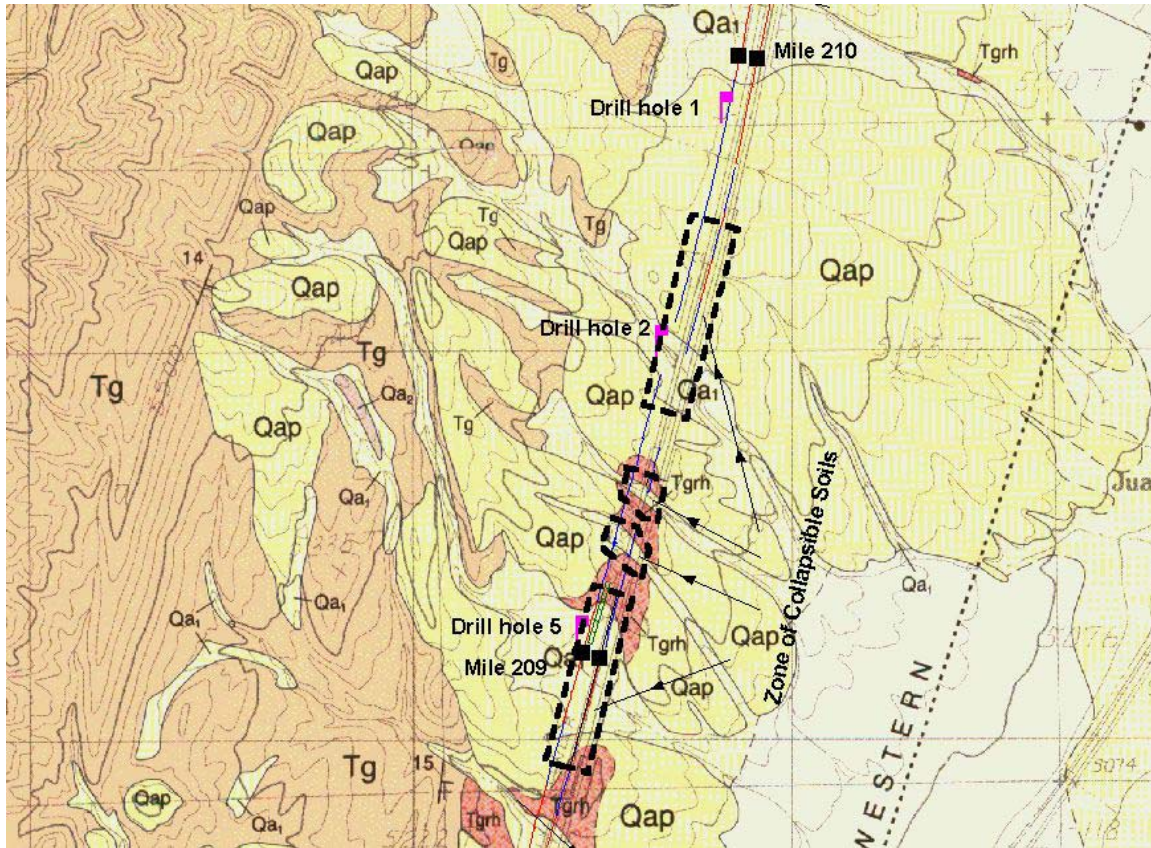


Figure ES-1: Map showing the four zones where collapsible soils have led to concentrated damage in the vicinity of milepost 209 and test holes 2 and 5. Lithology in the drainage basins upstream from the alluvial fans is conducive to the formation of collapsible soil.

Expansive Soil Treatment Recommendations

Distress related to expansive soils exists throughout the study area, but significant damage concentrations are located in a cut section between mileposts 208 and 207 along I-15. This area is long enough to propose treatments for the area, in order to improve ride quality throughout the cut section. Other areas are isolated and are generally too minor to be considered for treatment proposals. The map in Figure ES-2 shows the expansive soil zone which is recommended for treatment along I-15 and also includes the distress features and drainage paths throughout the cut section. The length of the treatment zone is 3209 feet in the Southbound lane (0.608 miles, Milepost 207.939 - 207.331) and 1779 ft in the Northbound lane (0.337 miles, Milepost 207.939 - 207.602).



Figure ES-2: Expansive soil zone proposed for treatment and the distress features throughout the cut section between mileposts 208 and 207 along I-15, Juab County, Utah.

A number of options for treating expansive soils are available; however, no single method has proven foolproof in practice. This study recommends a combination of methods as illustrated in Figure ES-3 to improve the odds of success. First, we recommend excavation of three feet of the expansive material and recompaction with the same soil treated with 5% lime. Laboratory tests indicate that lime treatment will significantly reduce the plasticity of the clay (from PI of 70 in untreated to 17 in treated soil) and increase the CBR (from about 5 in untreated to 50 in treated soil). Second, we recommend a continuous rubber asphalt layer which would extend under the drainage ditches on either side of the interstate. This layer would prevent infiltration of water into the subgrade which has occurred with the current surface drainage system. This impervious layer would need to be covered with a six inch layer of soil to protect it from damage. Finally, we recommend that the base courses be placed above the liner and that an asphalt wearing surface be used rather than concrete to minimize the potential for cracking.

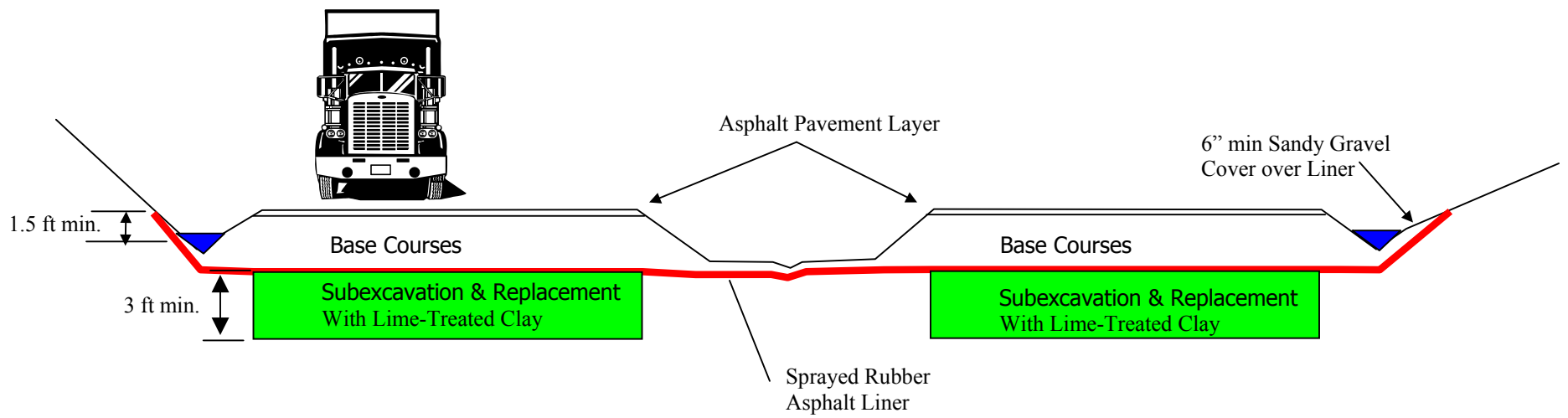


Figure ES 3: Schematic drawing of typical cross-section in expansive soils treatment zone.

Overall Recommendations

Pavement Type and Features

Because of the potential for differential settlement on the roadway, we recommend that asphalt pavement be used in reconstructing the roadway in the study area. Asphalt pavements provide several advantages relative to concrete pavements when expansive and collapsible soils are encountered. First, the pavement provides a “membrane” that helps restrict the infiltration of water into the subgrade. Second, if water does penetrate into the subgrade, the asphalt pavement is more flexible and is better able to accommodate the distortion without significant pavement distress. Third, the remedial repair of a damaged asphalt pavement can be completed quicker and easier than for concrete pavements. However, for highly moisture sensitive soils such as those encountered during this study, the use of asphalt paving alone will be insufficient to prevent pavement distress without the use of moisture barriers and subsurface treatment.

AASHTO guidelines suggest a 10-ft right shoulder and a 4-ft left shoulder. However, the further the infiltration and wetting surface can be maintained from the travel and passing lanes, the less likelihood of damage to the pavement (Snethen, 1979). Although the 10-ft right shoulder width is sufficient, the 4-ft left should probably be increased to a width of 6 to 8-ft.

Uniformity at Subgrade Discontinuities

Special care should be taken to assure that the subsurface characteristics are more uniform at discontinuities such as cut-fill transitions and around culverts. Within this study, a significant number of bumps and other distress features were located close to these zones. At cut-fill transitions within this study area, significant differences exist in unit weight and compressibility since the cuts tend to be in weathered rock while the fill sections are located on soil.

Minimization of the differences in physical characteristics is the simplest approach to reducing the localized distortions (Snethen, 1979). The subgrade in the transition to the cut section should be ripped or scarified (water added if necessary) and recompact to conditions comparable to the fill sections. A minimum depth of 12 inches should be considered, but preferably, the depth will be between 18 and 24 inches.

Around culvert or in utility or pipeline trenches, the backfill should never consist of coarse-grained material in expansive soil subgrades. Ideally, the backfill material should be a non-expansive cohesive soil compacted to a sufficient degree to minimize moisture infiltration into the trench. If the ideal material is not available, then the natural soil may be used provided it is thoroughly remolded and compacted at a higher moisture content. Consideration may also be given to using lime-stabilized soil.

Drainage Provisions

The lack of adequate surface drainage is one of the critical factors leading to problems with both collapsible and expansive subgrade soils. Some obvious signs of drainage problems include water ponding in the drainage ditches, soft spots in the ditch, or the presence of plants and weeds that grow best in saturated or submerged environments. These warning signs are present at a number of locations within the study area as noted previously in the section on site

investigations. We recommend that these drainage ditches be lined with asphalt with a protective covering of gravel to prevent leakage. In addition, we recommend that cross-drains which pass through the median be designed so that water does not accumulate in the median prior to passing through to the other side of the roadway as was observed at several locations.

PAVEMENT & SUBGRADE DISTRESS STUDY- REMEDIAL STRATEGIES FOR CONSTRUCTION AND MAINTENANCE

INTRODUCTION

Sections within a 17 miles length of roadway from about mileposts 217 to 200 south of Nephi, Utah have been experiencing considerable distress since construction. Maintenance costs have been significant and it appears that distress may not simply be due to an inadequate pavement section in all cases. The problems associated with bumps, cracks and edge failures are likely associated with problems in the subgrade soils along the alignment. Potential causes could include collapsible soil, expansive soil, compressible soil, poorly compacted fill and poor drainage. The objectives of this research study are to determine the causes for the problems and potential solutions prior to design and reconstruction of the roadway.

The results of this study are discussed in the following sections: (1) Identification of pavement distress areas, (2) Potential causes of pavement distress at various locations, (3) Subsurface Exploration , (4) Design approaches for mitigating the distress. This information will be useful to UDOT roadway design and maintenance personnel in the future.

1. IDENTIFICATION OF PAVEMENT DISTRESS AREAS

The first step in the study was to better define the locations where pavement distress was occurring. To accomplish this objective, we conducted a detailed mapping study using GPS units to identify areas where significant cracking, slumps or bumps and other pavement distress features are evident on the roadway alignment between mileposts 217 and 200. The locations of the pavement distress features were entered into an ARCINFO GIS map of the area.

Between mileposts 217 and 200 there are 119 distress features. Distress features include bumps, significant longitudinal and transverse cracking, and edge failures. Cracking is combined with edge failures for this study as simply edge failures. Three USGS geological maps were used to help in identifying where the problems are along I-15. The maps used were the Juab Quadrangle (1:24,000 scale), Skinner Peaks (1:100,000 scale), and the Mills Quadrangle (1:24,000 scale)

The number of edge failures and bumps within each mile of the study area is shown in Figure 1. The percentage of the total bumps, cracks and edge failures between each mile post is also shown in Figure 2. Over 49% of the 119 points occur between mileposts 210 and 207. Within these three miles there are 47 bumps and 12 edge failures/cracks. The majority of the bumps are between mileposts 217 and 207 and the majority of the edge failures/cracks are between mileposts 208 and 200.

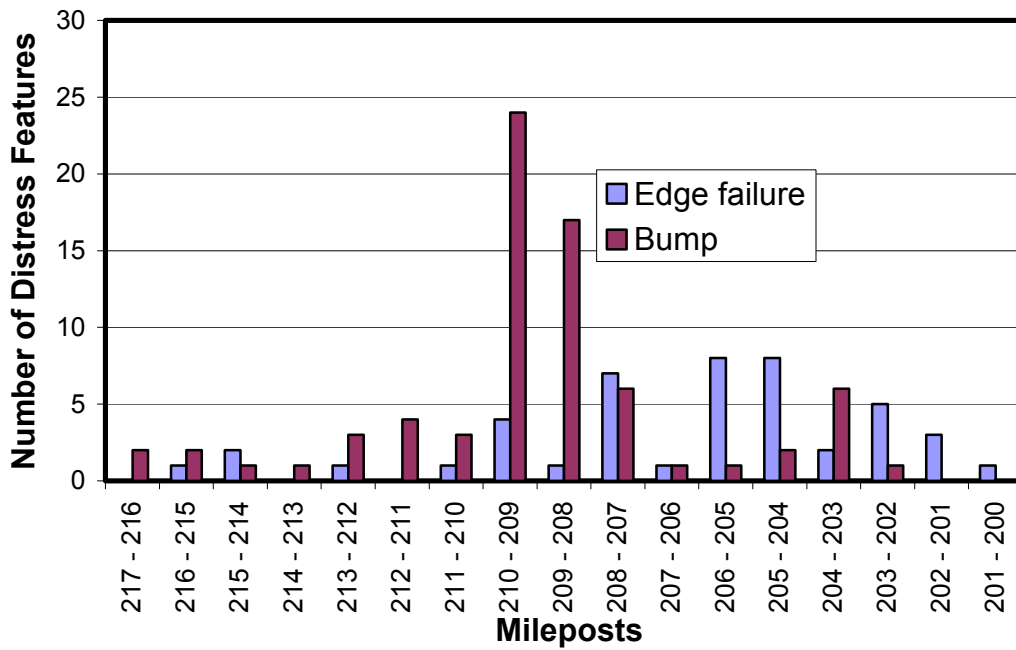


Figure 1: Points of damage including bumps and edge failures (includes cracks) vs. Mileposts; I-15 between Mileposts 217 and 200, Juab County, Utah, 2000.

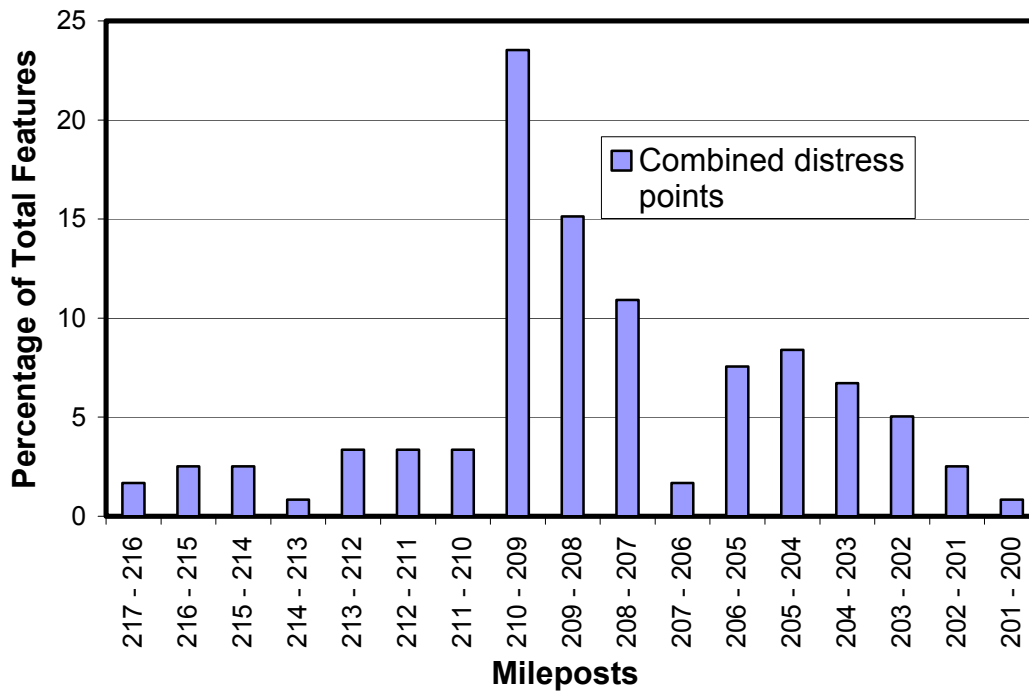


Figure 2: Combined percentage of distress points vs. Mileposts in between mileposts 217 and 200, I-15, Juab County, Utah, 2000.

2. POTENTIAL CAUSES OF PAVEMENT DISTRESS AT VARIOUS LOCATIONS

The geology and soil conditions along the length of I-15 in the study area are relatively complicated. As a result, the observed distress features may result from a number of potential causes. These potential causes include: (1) Collapsible soils within the stream alluvial deposits, (2) Moderate to highly expansive soils derived from weathered shales, claystones, and igneous tuffs, (3) Poor drainage along the side of the highway and crossing under the highway, (4) Abrupt cut/fill transition zones, (5) Inadequate pavement sections, and (6) Poorly compacted fill material. Most distress likely results from a combination of these causes, however, some may be a result of just one cause.

To better understand the causes of the problem, GIS based maps were produced using Arcview 3.2 in order to determine: (1) What the local subsurface geology is under each distress feature, (2) Where collapsible and expansive soils exist, (3) How many distress features are within 100 feet of a cut/fill transition zone, (4) How many distress features are within 100 feet of a culvert or drainage path, and (5) What distress features are near each of the Drill Holes along I-15. Available geological mapping and existing test hole logs have been used in this process along with supplemental drilling, soil sampling and laboratory testing.

Correlation with Surficial Geology

To help determine which distress features corresponded to which causes, the distress point locations obtained from the GPS mapping were first overlaid onto the surficial geologic maps for the study area. Three USGS geologic maps were used to help identify surficial geologic conditions in the study area along I-15. The maps used were the Juab Quadrangle map (1:24,000 scale), the Skinner Peaks map (1:100,000 scale), and the Mills Quadrangle map (1:24,000 scale). Figure 3 is a 3-D view of the combined geologic maps used for this study. The 3-D map helps to visualize where each map is located along with the topographic features in the vicinity. The z-axis (elevation) has been exaggerated in order to better define the low hills from the basins. Figures 4, 5, and 6 provide 2-D views of where the distress points are located on the Juab, Skinner Peaks, and Mills maps, respectively. The maps also include the locations of mileposts along I-15, Drill Holes drilled by UDOT for the project, and the drainage paths flowing under the freeway through culverts.

There are nine different geologic units that are related to the distress features. The encountered geologic units from north to south are Qacf, Qa1, Qap, Trgh, Tvg, Tg, Qtaf, Qtab, and Qtas. Because each geologic map was prepared by a different set of researchers, different symbol types have been used for similar geologic units on the maps. For example, the geologic units Qdf (Mills Quadrangle) and QTcf (Skinner Peaks Quadrangle) are the same as the geologic unit Qacf (Juab Quadrangle). The geologic unit Qal (Mills and Skinner Peaks Quadrangles) is the same as the geologic unit Qa1 (Juab Quadrangle). Qacf are coalesced alluvium fan deposits consisting of unconsolidated to semiconsolidated silts, sands, gravels, pebbles, cobbles, and sparse boulders. Qa1 are younger alluvium deposits consisting of clay to boulder-sized detritus, locally derived and deposited along intermittent streams. Qap are younger pediment alluvium deposits consisting of thin gently sloping mantles of clay to boulder-sized materials overlying a truncated bedrock surface. Trgh is the Hall Canyon Conglomerate (member of the Goldens Ranch

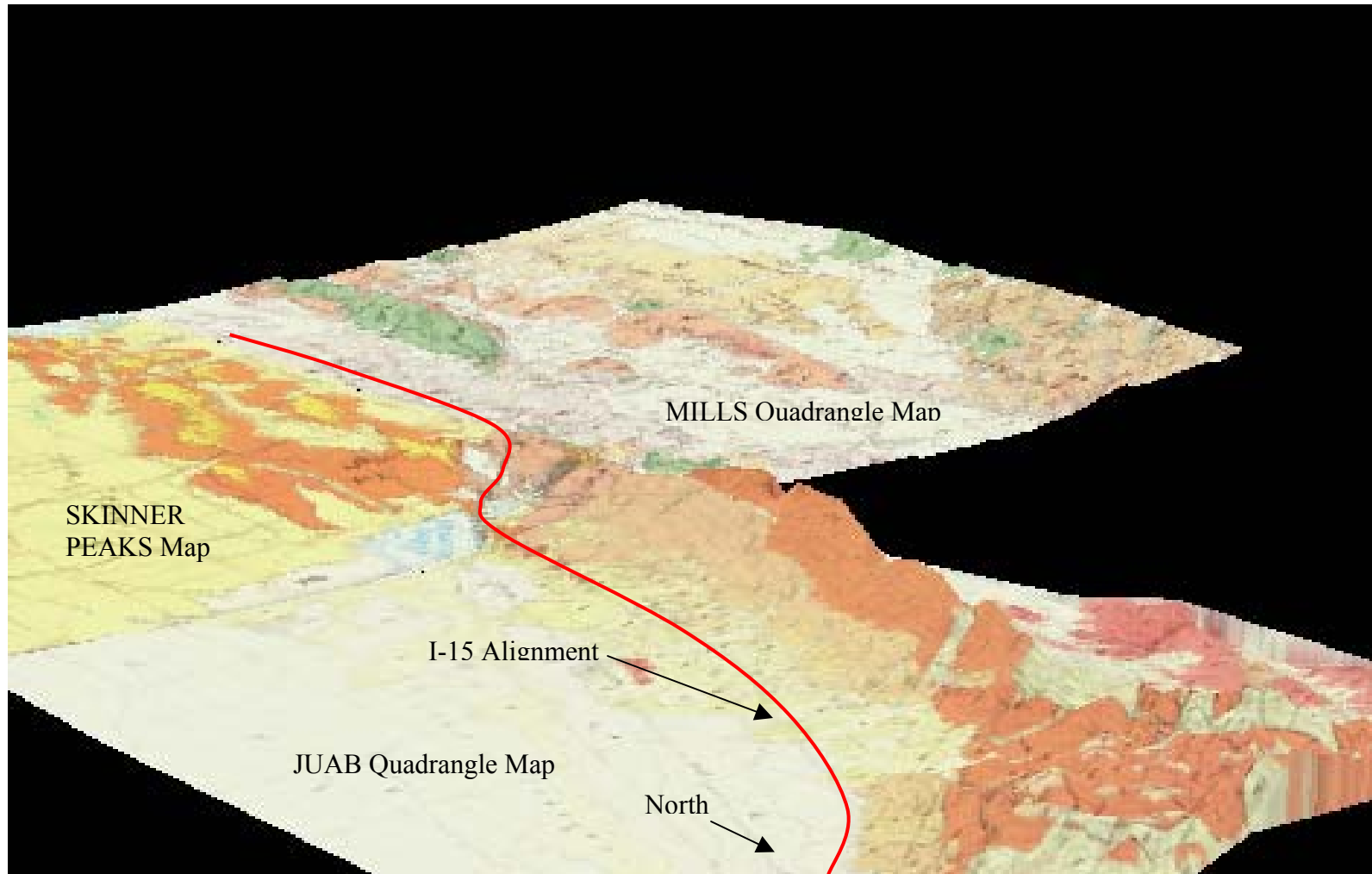


Figure 3: Combined 3-D view of Juab, Skinner Peaks, and Mills geologic maps. I-15 runs along the eastern base of the elevated hills on the Juab quadrangle.

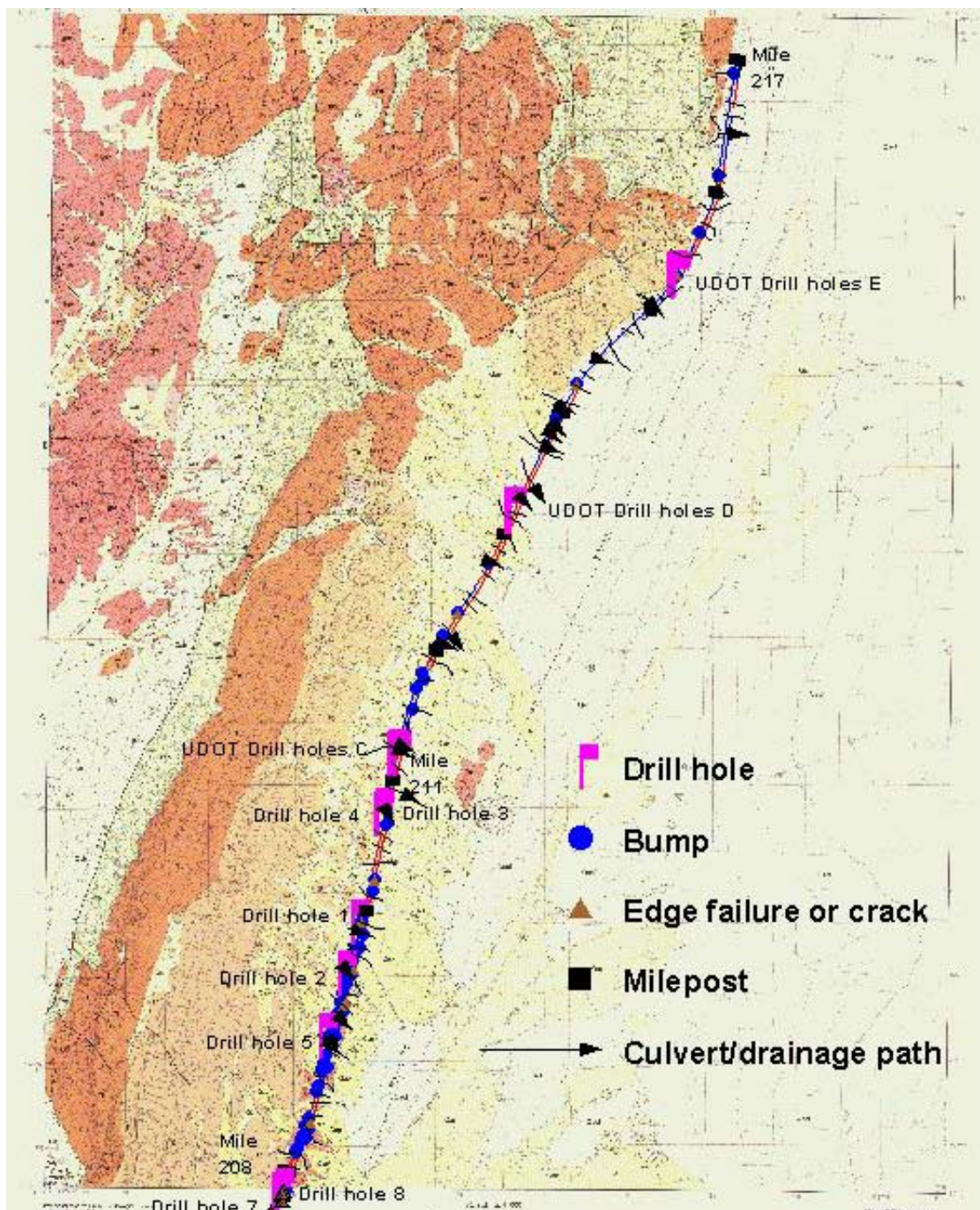


Figure 4: Map of Juab Quadrangle showing distress features, Drill Holes, mileposts, and where drainage paths flow under the freeway through culverts along I-15, UT.

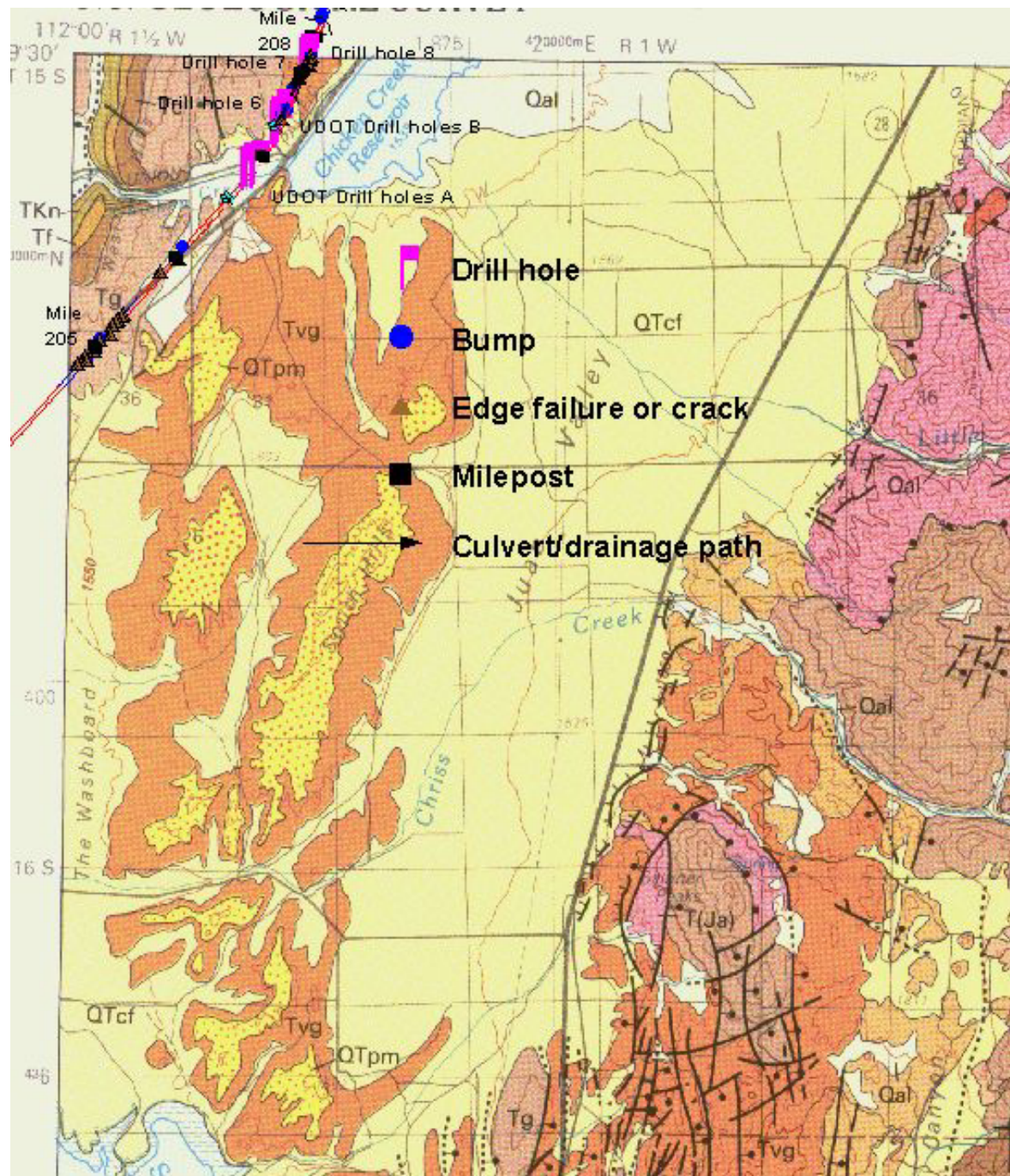


Figure 5: Map of Skinner Peaks area showing distress features, Drill Holes, mileposts, and where drainage paths flow under the freeway through culverts along I-15, UT.

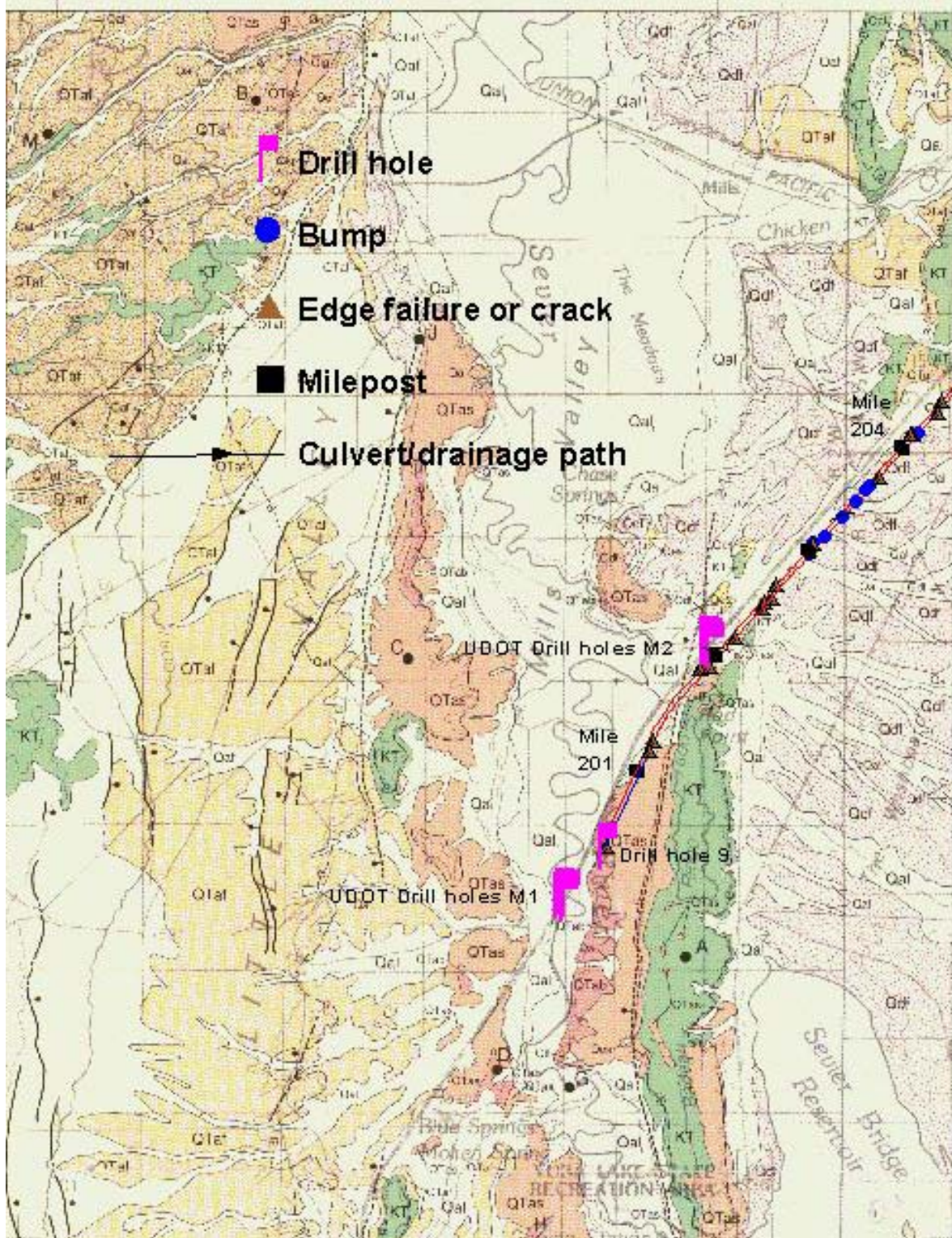


Figure 6: Map of Mills area showing distress features, Drill Holes, mileposts, and where drainage paths flow under the freeway through culverts along I-15, UT.

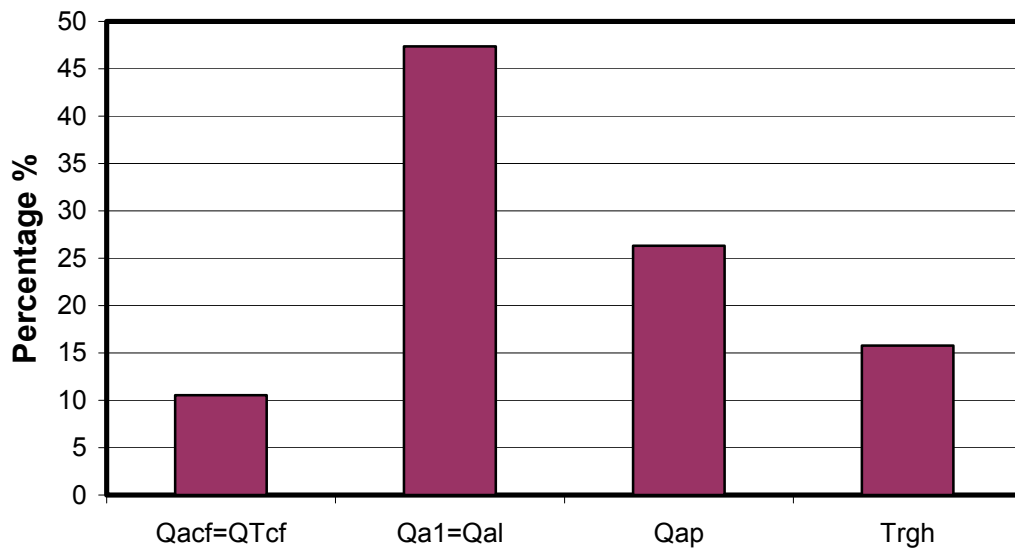


Figure 7: The percentage bump features underlain by various surficial geologic units in the Juab Quadrangle, UT. The percentage is based on a total of 57 bumps.

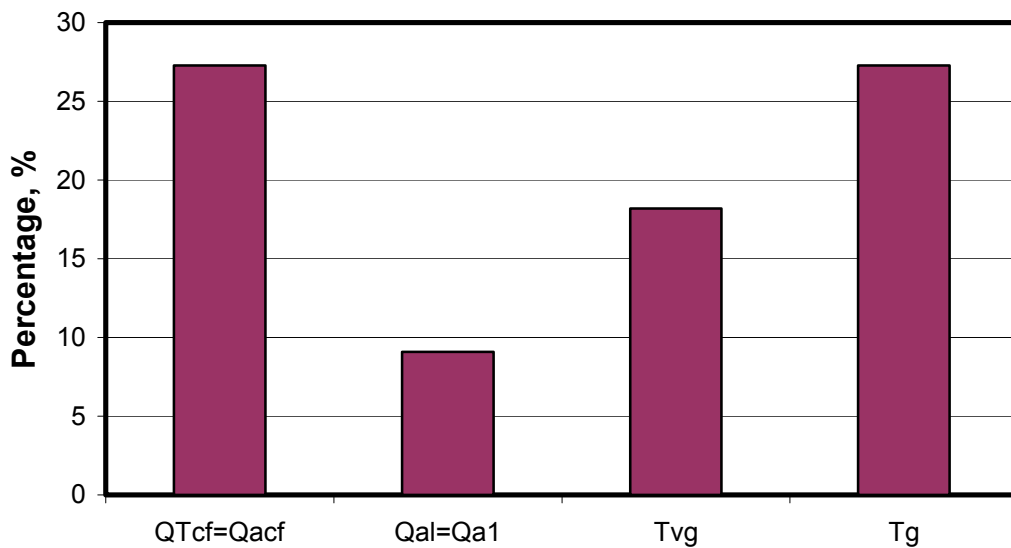


Figure 8: The percentage of bump features underlain by various surficial geologic units in the Skinner Peaks Quadrangle, UT. The percentage is based on a total of 9 bumps.

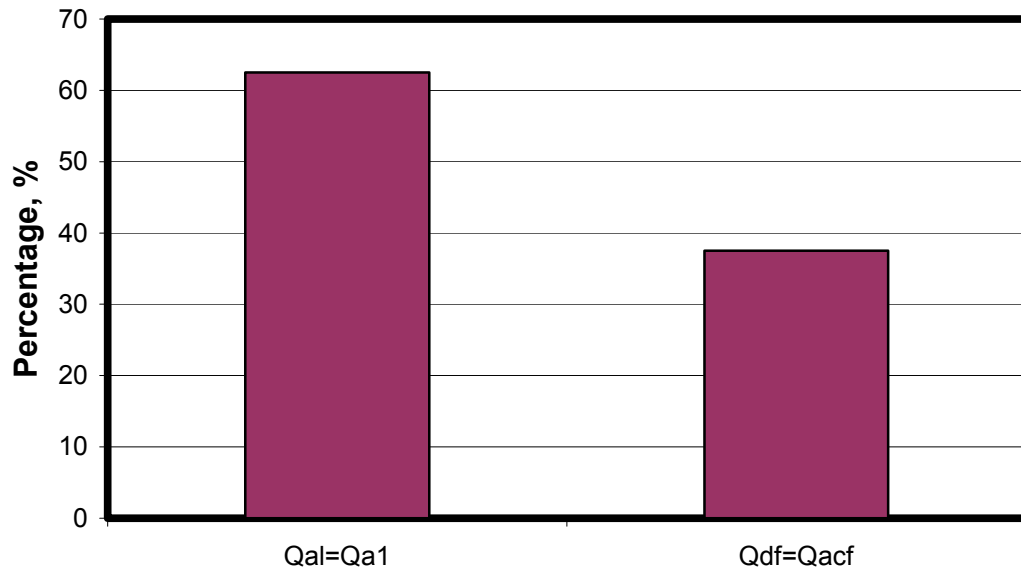


Figure 9: The percentage of bump features underlain by various surficial geologic units in the Mills Quadrangle, UT. The percentage is based on a total of 8 bumps.

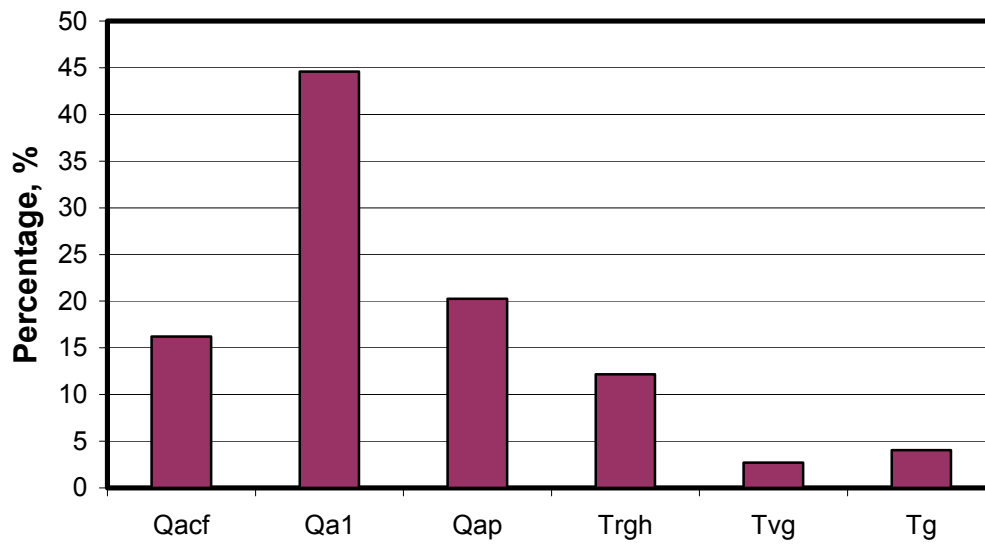


Figure 10: The percentage of bump features underlain by various surficial geologic units between mileposts 217 and 200. The percentage is based on a total of 74 bumps.

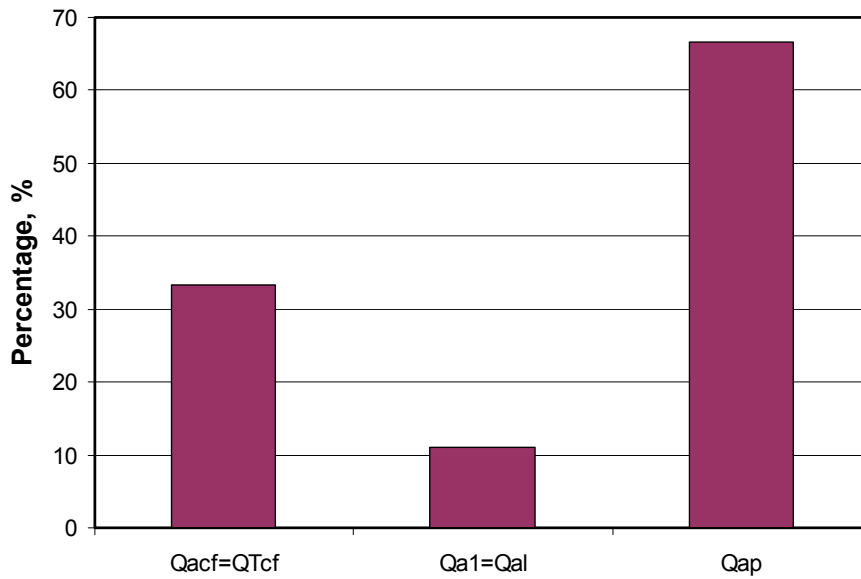


Figure 11: The percentage of crack/edge failures underlain by various surficial geologic units in the Juab Quadrangle, UT. The percentage is based on a total of 10 crack/edge failures.

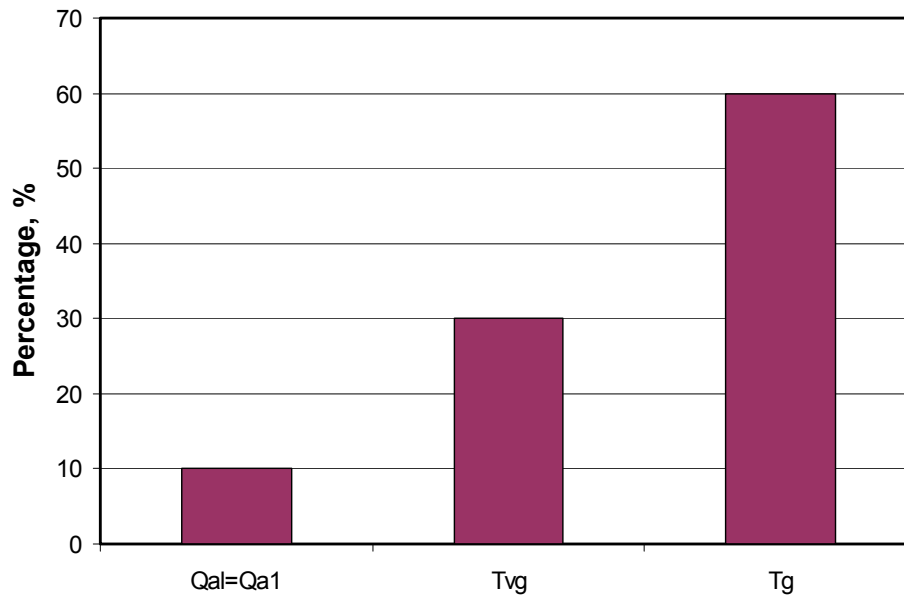


Figure 12: The percentage of crack/edge failures underlain by various surficial geologic units in the Skinner Peaks Quadrangle, UT. The percentage is based on a total of 20 edge failures.

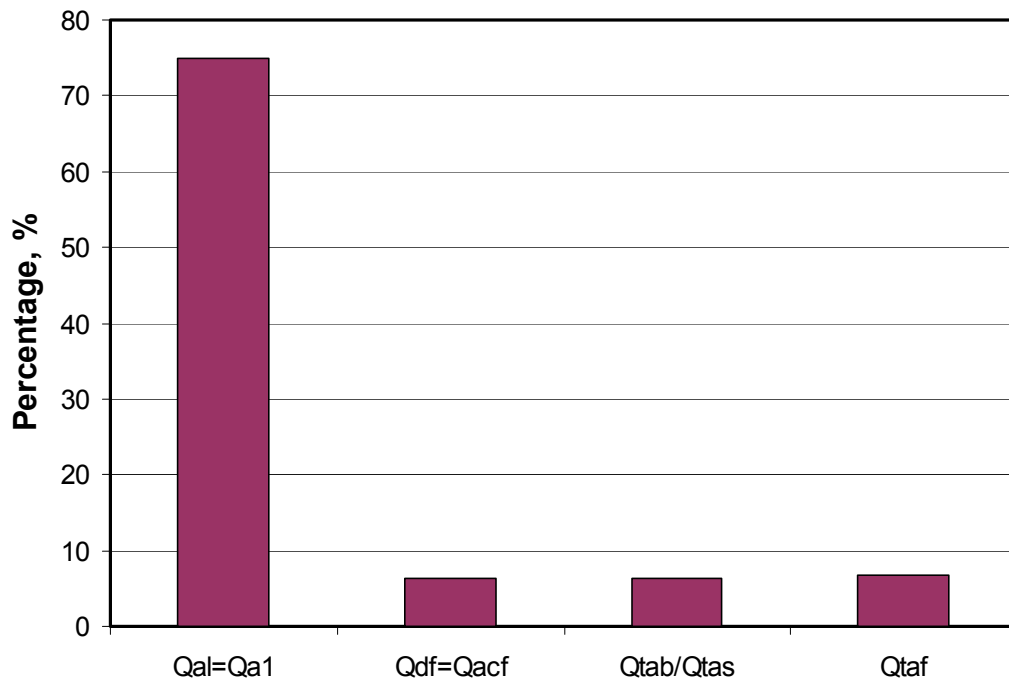


Figure 13: The percentage crack/edge failures underlain by various surficial geologic units in the Mills Quadrangle, UT. The percentage is based on a total of 16 edge failures.

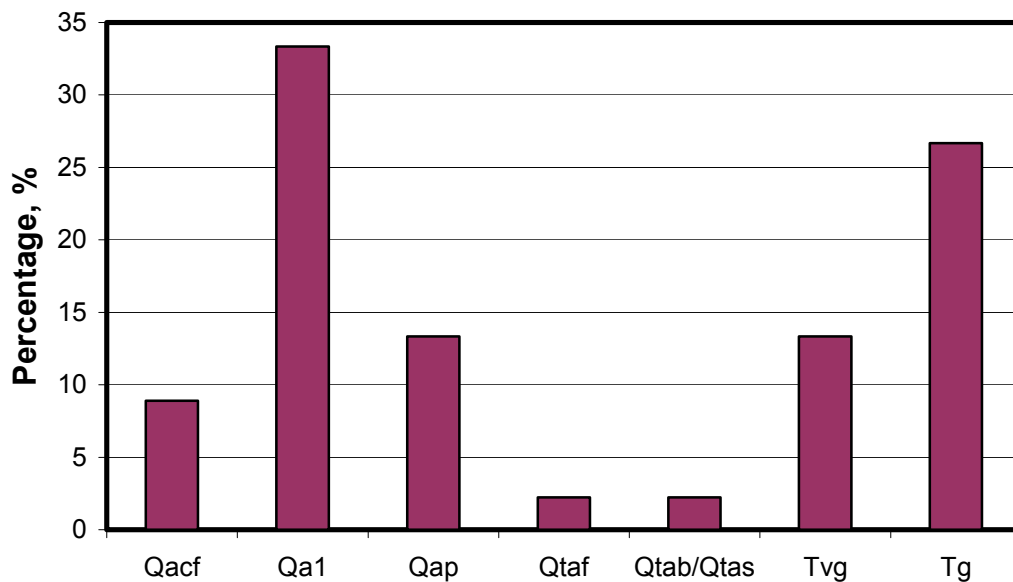


Figure 14: The percentage of crack/edge failures underlain by various surficial geologic units between mileposts 217 and 200. The percentage is based on a total of 46 edge failures.

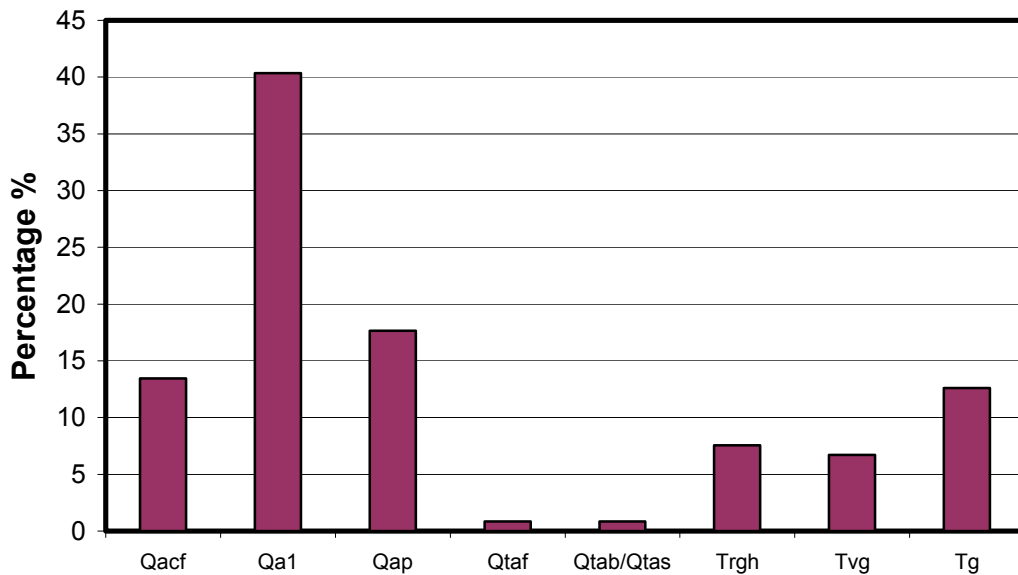


Figure 15: The percentage of total distress features within various surficial geologic units between mileposts 217 and 200. The percentage is based on 120 distress features.

Formation, Tvlg) and consists of gray-colored volcanic conglomerate, conglomerate, volcanic sandstone and siltstone, tuffaceous sandstone, tuff, and bentonitic clay. Tvlg is the Golden Ranch Formation of Muessig and consists chiefly of conglomerate and sandstone, and friable, light-pink and very light gray tuffs plus a lenticular, light-gray limestone. Tg is the Green River Formation and consists of a limestone unit underlain by a shale unit with thin sandstone and tuff layers.

The greatest percentage of distress features, nearly 40 %, fall within zones mapped as Qa1 materials. Thereafter, the percentage of distress points in various geologic units then decreases following the order of Qap, Qacf, Tg, Trgh, Tvlg, Qtaf, and Qtab/Qtas. Qtab/Qtas units are grouped together, because there is a distress feature that is on the borderline between both geologic units. Figures 7 through 10 illustrate the percentages of bump features corresponding to the various geologic units in each of the map boundary and for the entire study area combined. Figures 11 through 14 present plots showing the percentage of crack and edge failure features associated with various geologic units for each map and the entire study area combined. Figure 15 is a graph of the percentage of total distress features which are associated with each geologic unit for the entire project area.

Collapsible and expansive soils both exist between mileposts 217 and 200 and appear to correlate with some of the distress features. For example, collapsible soils are related to the geologic units Qacf, Qa1, and Qap. Qa1 and Qap are strongly connected with observed distress between mileposts 210 and 208 and account for 38 or 31.9% of all distress features. Most of the collapsible soil problems are in the Juab Quadrangle (Fig. 4). Overall, about 71% of the distress points fall in either the Qacf, Qa1, or Qap units throughout the study area and may be related to collapsible soils.

Distress features that relate to expansive soils are found within two main cut sections. One cut section is near Drill Hole 1 and milepost 210 (Fig. 4). The other cut section is between Drill Holes 6 and 8, which is between mileposts 208 and 207 (Fig. 5). Expansive soils appear to be associated with units Trgh, Tvg and Tg. Distress features within these units account for 26% of the distress features overall. Out of the 14 distress points located in the Tvg unit, which has a high potential for expansive materials, 13 of them are within the second cut section between mileposts 208 and 207. A bar graph is shown in Figure 16 which indicates the percentage of distress features, which correlate with the collapsible soils and the expansive soil units.

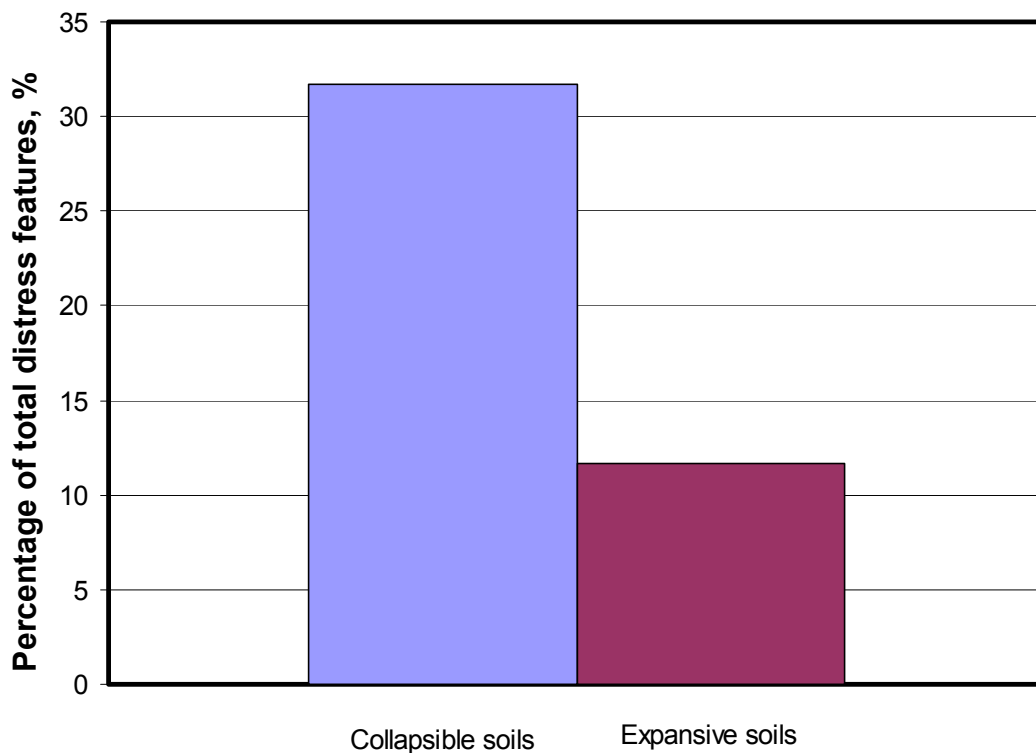


Figure 16: Percentage of distress features between mileposts 217 and 200 that are related to either a collapsible soil between mileposts 208 and 210 or an expansive soil between mileposts 207 and 208.

Correlations with Cut/Fill Sections

A total of 20 or 16.8 % of the distress features fall within 100 feet of a cut/fill transition zone and therefore could be related to soil density contrasts between the cut section and the fill section. A total of 18 or 26.9 % of the distress features in the Juab Quadrangle (Fig. 4) occur within 100 feet of a cut/fill transition zone. Only 2 or 6.9 % of the distress features in the Skinner Peaks Quadrangle (Fig. 5) occur within 100 feet of a cut/fill transition zone. The Mills Quadrangle (Fig. 6) has no distress features that occur within 100 feet of a cut/fill transition zone.

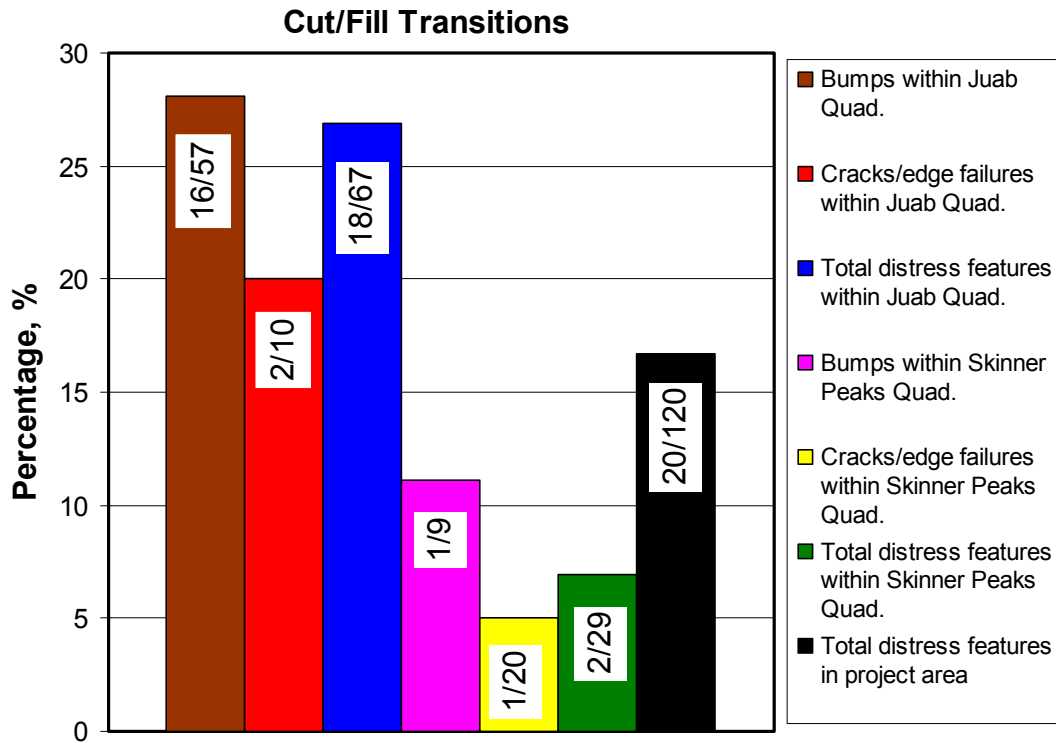


Figure 17: Percentage of bumps, edge failures, and distress features that are within 100 feet of a cut/fill transition zone for Juab Quadrangle, Skinner Peaks Quadrangle, and the entire study area between mileposts 217 and 200.

The bar graphs in Figure 17 show the percentage of bumps, edge failures, and the total number of distress features within 100 feet of a cut/fill transition zone as a percentage of the total number of bumps, edge failures, and distress features for the Juab Quadrangle, Skinner Peaks Quadrangle, and the entire study area between mileposts 217 and 200. Figure 18 represents the percentage of bumps or edge failures that occur in either a cut or fill section along I-15 between mileposts 217 and 200. Although the majority of the distress features (57%) are in cut sections, a significant percentage (43%) still occur in fill sections. Therefore, the distress does not appear to be simply a matter of poor fill construction. Our analysis also indicates that the potential for distress does not appear to be correlated with the height of the fill. In fact, in areas where the fill is relatively high, the pavement performance has been relatively good.

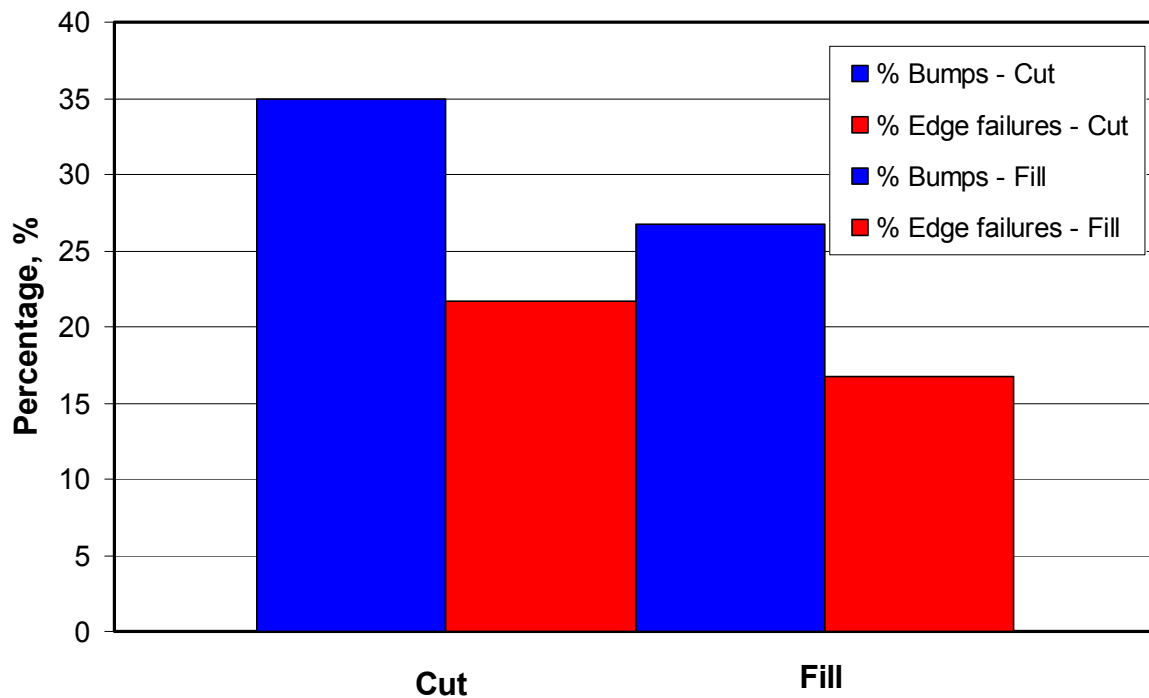


Figure 18: Percentage of total bumps or edge failures within a cut and fill section between mileposts 217 and 200 along I-15, Juab County, Utah.

Correlation with Culvert or Drainage Features

A total of 42 (35.3 %) distress features fall within 100 feet of a culvert or drainage path. Drainage problems can include inadequate compaction over the culvert and the expansion or collapse of soils from excess water permeating into the subsurface materials. 29 or 43.3 % of the distress features in the Juab Quadrangle (Fig. 4) occur within 100 feet of a culvert or drainage path. 13 (44.8 %) of the distress features in the Skinner Peaks Quadrangle (Fig. 5) occur within 100 feet of a culvert or drainage path. The Mills Quadrangle (Fig. 6) has no distress features within 100 feet of a culvert or a drainage path. The bar graphs in Figure 19 show the percentage of bumps, edge failures, and the total amount of distress features within 100 feet of a culvert or a drainage path as a percentage of the total number of bumps, edge failures, and distress features for the Juab Quadrangle, Skinner Peaks Quadrangle, and the entire study area between mileposts 217 and 200.

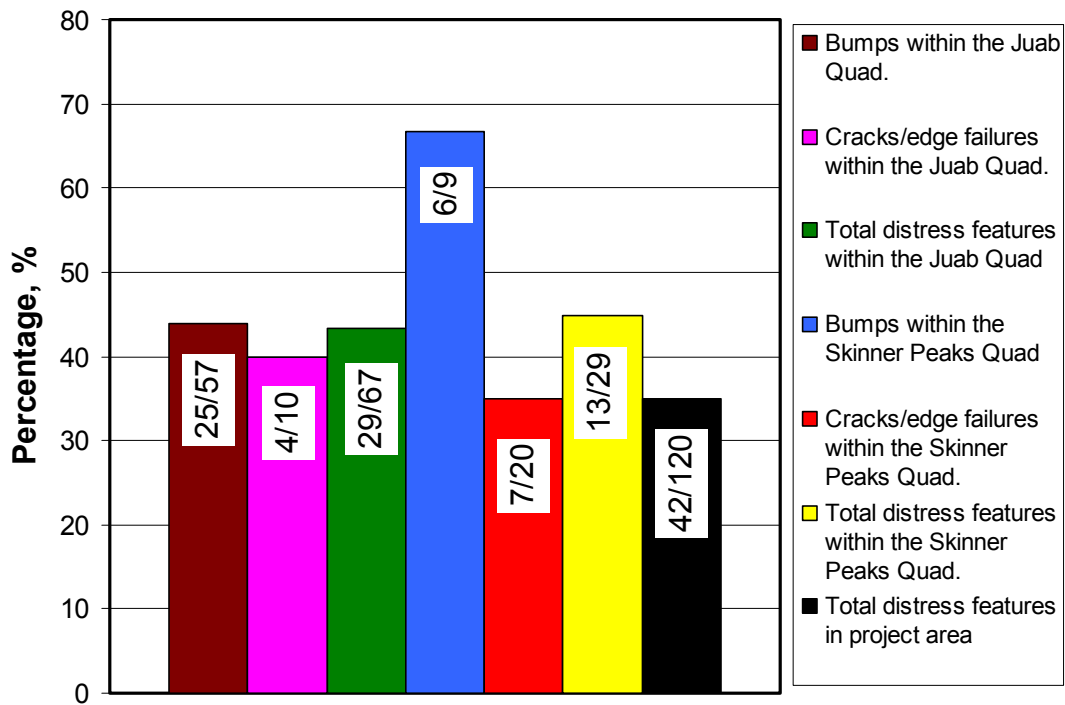


Figure 19: Percentage of bumps, edge failures, and distress features that are within 100 feet of a culvert or a drainage path for the entire study area between mileposts 217 and 200.

3. SUBSURFACE EXPLORATION

To better determine the characteristics of the subsurface geology and its relationship to the distress features seen between mileposts 217 and 200, nine test holes were drilled in August 2000 by UDOT personnel. Previous drilling was done during the late 1970's and early 1980's for seven different structures built along I-15 between mileposts 216 and 199. Four test holes were drilled for each of the seven structures investigated. Test data and borelogs from the August 2000 drill holes and the boring logs from the earlier UDOT Drill Holes will be used together to correlate between the subsurface geology and the distress features. The soil conditions will be grouped together and discussed for sections moving from the north to the south of the study area.

Soil Conditions Near UDOT Drill Holes E

These four holes were drilled for the overpass structure of Lampson Canyon Road at the northern end of the study area about 0.4 miles north of milepost 215. The boring logs for the four holes are shown in Figure 20. No significant pavement distress features were noted in this region. Although these holes were drilled in the Qacf geologic unit, the boring logs indicate that the surficial alluvial sandy silt layer is only 1.5 to 5 feet thick. This alluvial sandy silt layer is typically underlain by a stiff clayey silt derived from weathering of the underlying claystones and siltstones. The plasticity index of the clayey silt is typically between 12 and 18 which



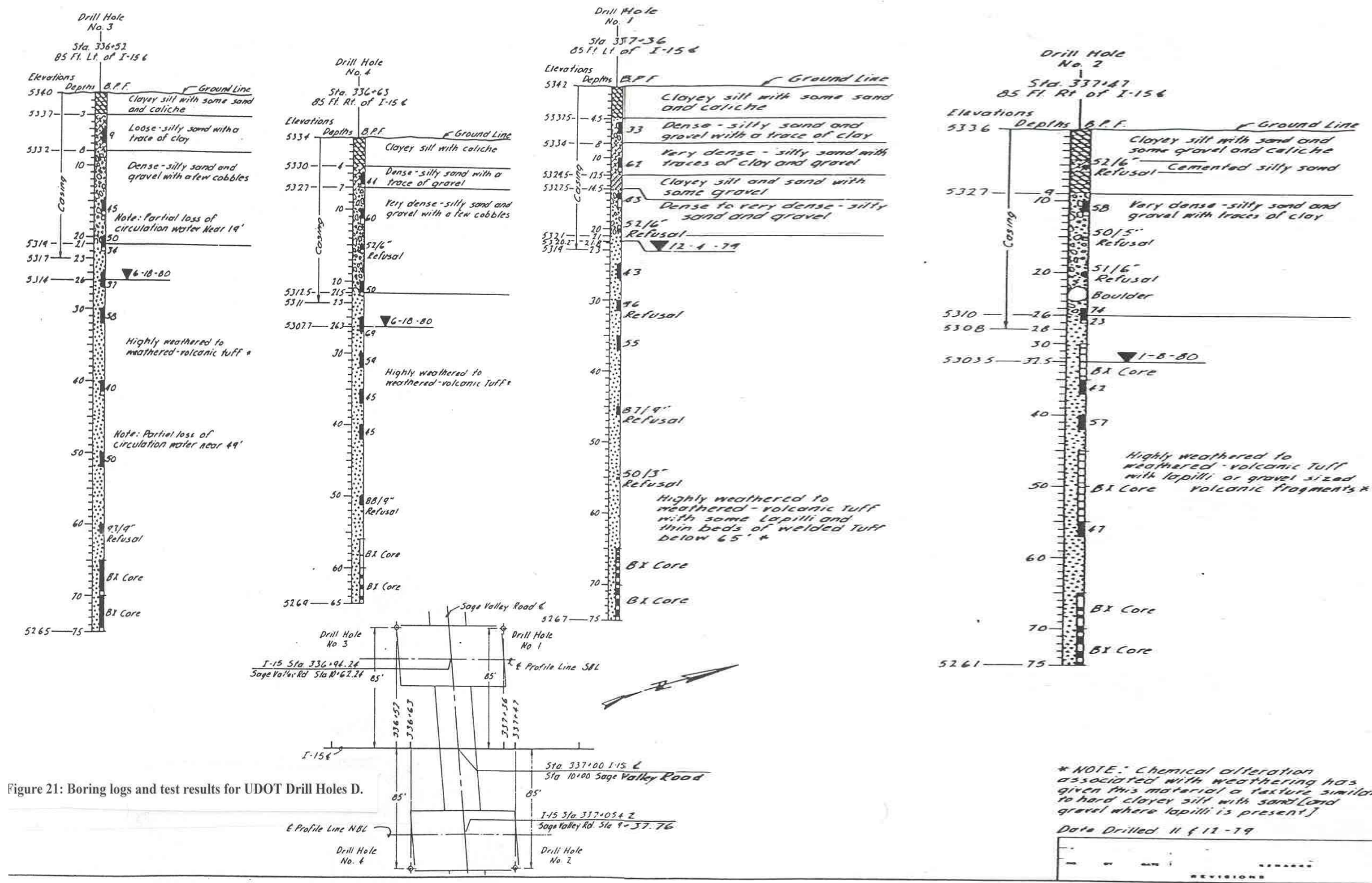


Figure 21: Boring logs and test results for UDOT Drill Holes D.

suggests that the potential for expansion is relatively low. This clayey silt layer is typically underlain by weathered claystone which becomes progressively stronger with depth. SPT tests within the upper 4 feet of the claystone generally reached refusal and at greater depths, coring was required to obtain samples.

No collapse test data is available for the alluvial silt layer. Although these layers may have some potential for collapse, they are relatively thin and are under relatively low vertical stress. In addition, there does not appear to be any significant ponding of water around the culverts and drainage channels which would wet the underlying soils. There are also no swell tests for the clayey silt layers, but the moderate plasticity suggests that expansion will not be a concern. Therefore, based on our review of the soil conditions at this site, the lack of distress features appears to correlate well with the relatively favorable soil conditions in this vicinity.

Soil Conditions Near UDOT Drill Holes D

These four holes were drilled for the overpass structure F-433 over the Sage Valley road, near the northern end of the study area about 0.2 miles north of milepost 213. The boring logs for the four holes are shown in Figure 21.

No significant pavement distress features were noted in this region. These holes are drilled in the Qacf geologic unit. The boring log indicates that the soil profile generally consists of a clayey silt layer, 3 to 5 ft thick underlain by a silty sand layer, 3 to 5 ft thick, which in turn underlain by a dense silty sandy gravel layer which extends to a depth of 20 to 26 feet below the ground surface. These alluvial deposits are then underlain by a highly weathered to weathered volcanic tuff which extends to a depth of 75 feet below the ground surface. The water table is typically located a few feet below the rock-soil interface.

The data from the boring logs indicate that the silty sands and silty sandy gravels are generally quite dense. SPT blow counts in the sands are generally between 33 and 62, while those in the sandy gravel are typically greater than 45 and often reach refusal. These high blow counts suggest that these materials are not likely to be collapsible, although some silty gravel materials have collapsed under relatively high pressures (Rollins et al, 1994). No collapse test data is available for the surficial silt layer. Although this layer may have some potential for collapse, the layer is relatively thin and the vertical stress is very low so the strain would likely be small. As in the case around Drill Holes E, there does not appear to be any significant problem with the surface drainage system in this vicinity. Based on our review of the soil conditions at this location, the relatively good pavement performance appears to correlate with the favorable soil conditions.

Soil Conditions Near Drill Holes 3, 4 and UDOT Drill Holes C

Drill Holes 3 and 4 were drilled as companion holes with Drill Hole 4 being about 100 feet south of Drill Hole 3. Drill Holes 3 and 4 are located about 1000 ft south of Milepost 211 along I-15. UDOT Drill holes C were located about 1000 ft north of Milepost 211 as shown in Figure 22. There are no distress features near either Drill Hole 3, 4 or UDOT Drill holes C. Boring logs for Drill Holes 3 and 4 are presented in Figures 23 and 24 while logs for UDOT Drill Holes C are shown in Figure 25. All of these holes are located in alluvial fan materials with geologic units defined as Qa1 or Qap. Drill Hole 3 is located in the fill section between the southbound and

northbound lanes of traffic. This hole was drilled to a depth of 15.5 feet through fill material, stream alluvium, and weathered shale/mudstone. Drill Holes 3 and 4 are located within geologic units Qa1 and Qap. To the west of Drill Hole 3 there is a cut section and to the east of Drill Hole 3 along the northbound lane there is a cut/fill transition zone. Drill Hole 4 is located about 100 ft south of Drill Hole 3 and about 20 feet away from the west side of the southbound lane of I-15 within a cut section. All the test holes are located near culverts or drainage channels.

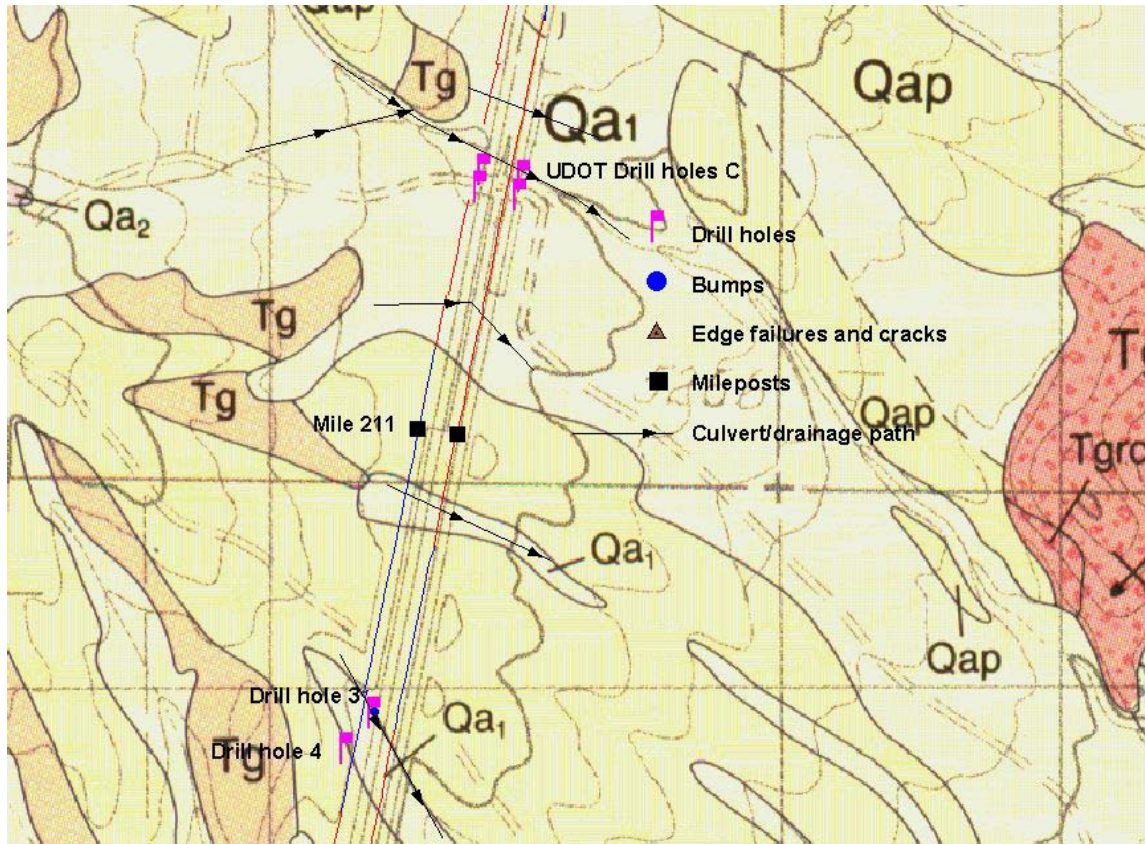


Figure 22: Drill Holes 3, 4 and UDOT Drill Holes C near milepost 211, I-15 Juab County, UT. Cuts are in blue and fills are in red.

Drill Holes 3 and 4 encountered sandy silt and silty sand layers from the alluvial fans which could be collapsible, but the layers are relatively thin and no evidence of collapse features were observed near the drainages. In fact, these alluvial soils and the fill materials above them generally have high SPT blowcounts and are relatively dense. These alluvial soils are underlain by high plasticity clays derived from weathering of the underlying claystone and volcanic tuff. These clays were encountered at a depth of 9.5 feet in Drill Hole 3, at 3.5 feet in Drill hole 4 and at 13 to 17 ft in the UDOT Drill Hole C location. These clays are likely expansive based on their high plasticity but again there does not appear to be any major pavement distress in this area. Apparently the relatively low permeability of the dense overlying material was sufficient to prevent wetting of the high PI materials or if the clays did become wet, the surcharge provided by the overlying soil prevented significant heaving.

LOG OF BORING, REV. 1 NEPHI.GPJ LANE GDT 4/10/01

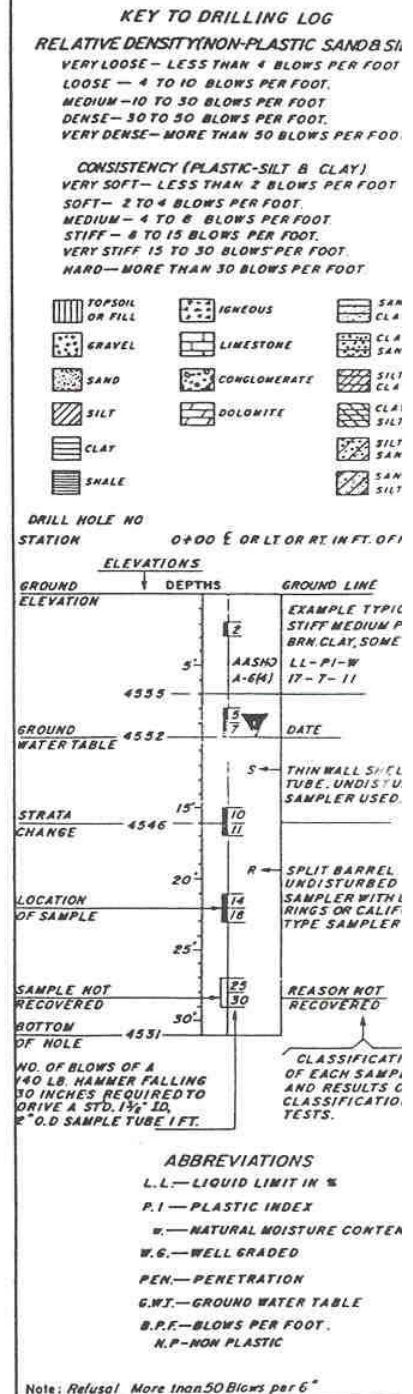
BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449				Boring No: BH-3 Logged By: Sizemore Date Started: 8/2/2000 Date Finished: 8/2/2000				Sheet 1 of 1 Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air				NEPHI I-15										
Location		Water Level	Depth (ft)	Graphic Log	Sample Data						% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS		
Material Description					Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N) ₆₀ Blows/ft						Soil Classification		LL			PL	PI
																USCS	AASHTO					
Sandy silt with gravel, some clay, fill, dry to damp, brown - medium dense					3-1.5	MC	18 15	12-17-31	48	CL	A-6				11.3	49	24	25	Shelby refusal at 3.5'			
sandy silt, very damp, brown					3-5	MC	24 15	10-12-11	23	SC	A-2-6	25	52	24	13.9	29	19	10				
sandy silt with gravel, some clay, brown, damp - medium dense			5		3-7.5	MC	18 13	11-19-31	50						13.3							
very silty clay with gravel, some sand, very damp, lt. brown					3-10.4	MC	18 15	15-37-63	refusa	CH	A-7-6				22.0	86	31	55				
highly weathered mudstone, yellow brown, grey and lt. grey, very damp to wet - refusal			10		3-13.5	SPT	18 16	17-37-50	refusa						25.1							
siltstone, lt. grey (yellow), damp, weathered - refusal			15		3-15.5	SPT	6 6	50	refusa						14.8							
											</											

LOG_OF_BORING_REV_1_NEPHI.GPJ LANE.GDT 4/10/01

LOG OF BORING, REV. 1, NEPHI.GPJ, LANE.GDT 4/10/01

BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449			Boring No: BH-4A Logged By: Sizemore Date Started: 7/26/2000 Date Finished: 7/26/2000		Sheet 1 of 1 Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air		NEPHI I-15																
Location			Water Level	Depth (ft)	Graphic Log	Sample Data						% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS		
Material Description						Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N ₁₆₀) Blows/ft						Soil Classification		Liquid Limit			Plastic Limit	Plasticity Index
																	USCS	AASHTO					
sandy silt, traces of clay, brown, dry				1				SPT	24	5-9-12	21	CL	A-6				10.2	44	23	21			
sandy silt, traces of clay and gravel, brown, dry				2																			
clayey silt, light grey to brown, dry to very damp, probably highly weathered shale - hard				3																			
				4		4A-4.5	MC		18 6	16-30-37	67	CH	A-7-6				22.1	88	27	61			
				5																			
weathered siltstone, grey, damp - refusal				6																			
				7		4A7.3	MC		15 11	24-53-60		CH	A-7-6				15.0						
				8																			
				9																			
				10																			
LEGEND AND NOTES			SPT = Standard Penetration Test, Split Spoon Sample US = Undisturbed Shelby Tube, Pushed Ring = Dames & Moore Sampler Bulk = Bulk Sample From Cuttings			GROUNDWATER DEPTHS: = Initial = End of Drilling			COMMENTS: Fig. 24 Boring log and test results for Drill Hole 4														

LOG OF BORING REV. 1 NEPHI.GPJ LANE.GDT 4/10/01



Note: Holes drilled 4th & 5th month - 1980

The samples from Drill Holes 3 and 4 were taken back to the Soil Mechanics laboratory at BYU and tested. Atterberg limit tests and grain-size distribution tests were both conducted on some of the samples. The results of the testing are summarized on the borehole logs for Drill Holes 3 and 4. The logs also include the classifications according to both the Unified Soil Classification System and the AASHTO classification system for those samples where enough information was available. The samples that did not have grain-size distribution curves were classified using their Atterberg limits and visual estimations of their grain-size distributions.

No consolidation tests were completed on the samples from Drill Hole 3. Two attempts were made to trim ring samples for collapse tests, however, there were too many gravel particles to be put in a consolidation test ring.

Grain-size distributions were determined by mechanical analysis on samples at 3.5 and 4.5 ft in Drill Hole 3 and the results have been plotted in Figure 26. A hydrometer test was also performed on the sample at 3.5 ft which showed that the fines were about evenly divided between silt and clay particle sizes. Both samples are fairly well graded and have a similar particle distribution with the sample at 3.5 ft being slightly more well-graded.

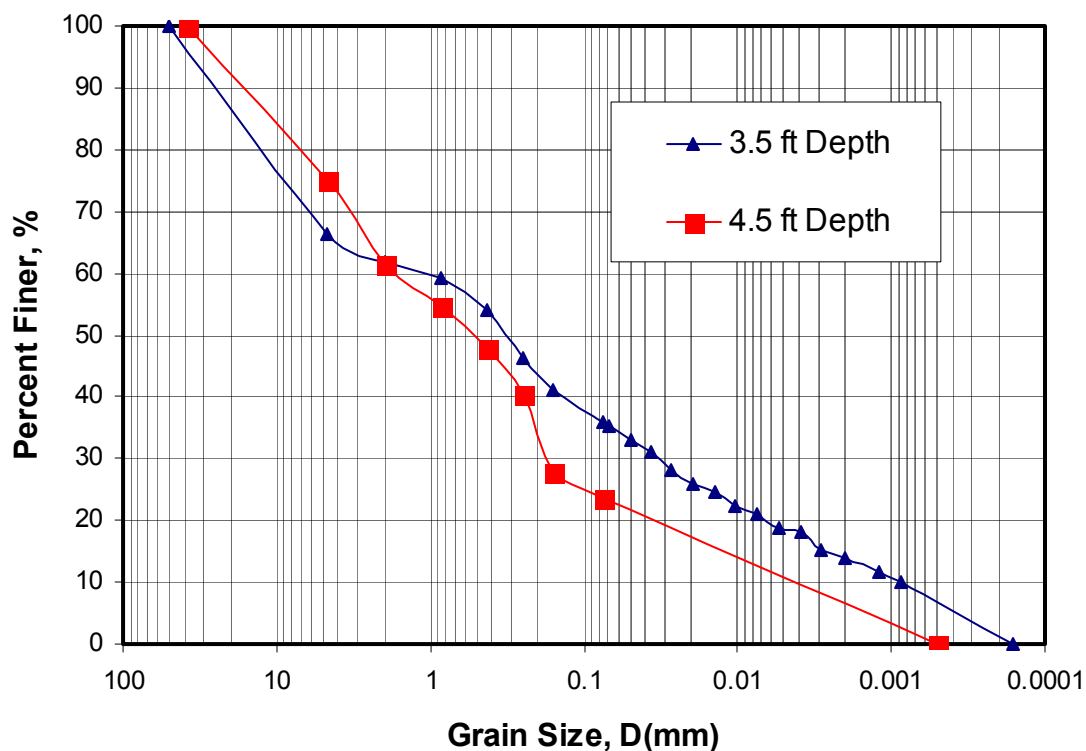


Figure 26: Grain-size distribution tests results for samples from Drill Hole 3.

Soil Conditions Near Drill Hole 1

Drill Hole 1 is located in the west cut section on the west side of southbound lane, just a few hundred feet south of milepost 210 as shown in Figure 27. Drill Hole 1 was drilled to a depth of 4.5 feet through alluvial fan deposits and into weathered shale/mudstone. The soil profile for the borehole log is shown in Figure 28. Drill Hole 1 is located within geologic unit Qap. Drill Hole 1 does have mildly expansive soils at 3.5 feet deep. Distress features possibly relate to a collapsible/expansive soils, cut/fill transition zones, and a lack of proper drainage. Figure 29 is a picture of a large longitudinal crack next to Drill Hole 1. This crack is likely related to the moderately expansive clays located in the area. One bump near Drill Hole 1 is located within 100 feet of a cut/fill transition zone. Another bump near Drill Hole 1 is located within 100 feet of a cut/fill transition zone and a culvert. One other bump near Drill Hole 1 is located within 100 feet of a culvert. The final six distress features (6 bumps and 1 edge failure) are possibly related to the subsurface soils under the highway.

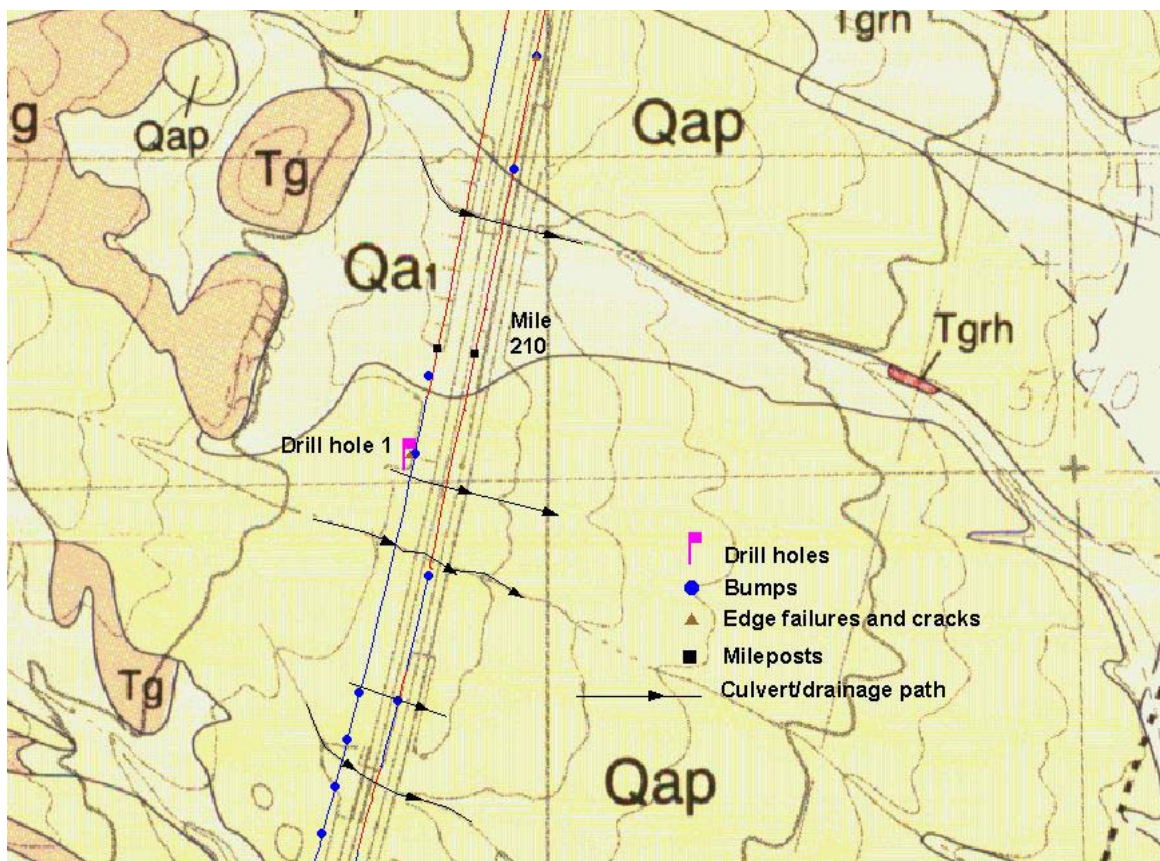


Figure 27: Drill Hole 1 and the distress features nearby, I-15, Juab County, UT. Cuts are in blue and fills are in red.








BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449			Boring No: BH-1		Sheet 1 of 1		NEPHI I-15																				
			Logged By: Sizemore		Elevation (ft):																						
			Date Started: 7/27/2000		Drill Contractor: UDOT																						
			Date Finished: 7/27/2000		Drilling Method: Rotary w/air																						
Location			Water Level	Depth (ft)	Graphic Log	Sample Data						% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS						
						Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N ₁₆₀) Blows/ft						Soil Classification		Liquid Limit			Plastic Limit	Plasticity Index				
																	USCS	AASHTO									
Material Description																											
Fat Clay				1		1-2A	SPT	$\frac{24}{18}$	9-8-8	16	CH	A-7-6	20	20	60		11.7	56	22	34							
Fat Clay				2																							
				3																							
				4		1-4.5	MC	$\frac{18}{9}$	17-31-52	83	CH	A-7-6	0	3	98	112.5	16.9	70	17	53							
Weathered Shale/Mudstone				5																							
				6																							
				7																							
				8																							
				9																							
				10																							
			MC=Modified California Sampler																								
LEGEND AND NOTES			 SPT = Standard Penetration Test, Split Spoon Sample  US = Undisturbed Shelby Tube, Pushed  Ring = Dames & Moore Sampler  Bulk = Bulk Sample From Cuttings										GROUNDWATER DEPTHS:  = Initial  = End of Drilling					COMMENTS: Fig. 28 Boring log and test data for Drill Hole 1									



Figure 29: Longitudinal crack near Drill Hole 1, I-15, Juab County, UT.

Samples from Drill Hole 1 were tested at BYU and the results from the Atterberg limit testing and grain-size distribution testing are shown on the boring log in Figure 28. Figure 28 also includes the classifications according to both the Unified Soil Classification System and the AASHTO classification system

A consolidation test was also performed on a ring sample trimmed from an undisturbed sample at 3.5 feet depth and the results are shown in Figure 30. This sample had a dry unit weight of 112.5 lb/ft^3 ($e_o=0.50$) and a natural moisture content of 16.9 %. The specimen was initially placed in the consolidometer at the natural moisture content and water was added when the pressure was approximately equal to the overburden pressure. The sample swelled about 1.0% when wetted. Swelling was expected based on the high PI of the soil. However, as the pressure increased to 22.4 ton/ft^2 , the specimen compressed 13.3 % (see Figure 30). The compression index (C_c) was 0.104. The grain-size distribution for this same sample is shown in Figure 31. The sample contained 98 % fines with 50 % silt and 48 % clay size particles.

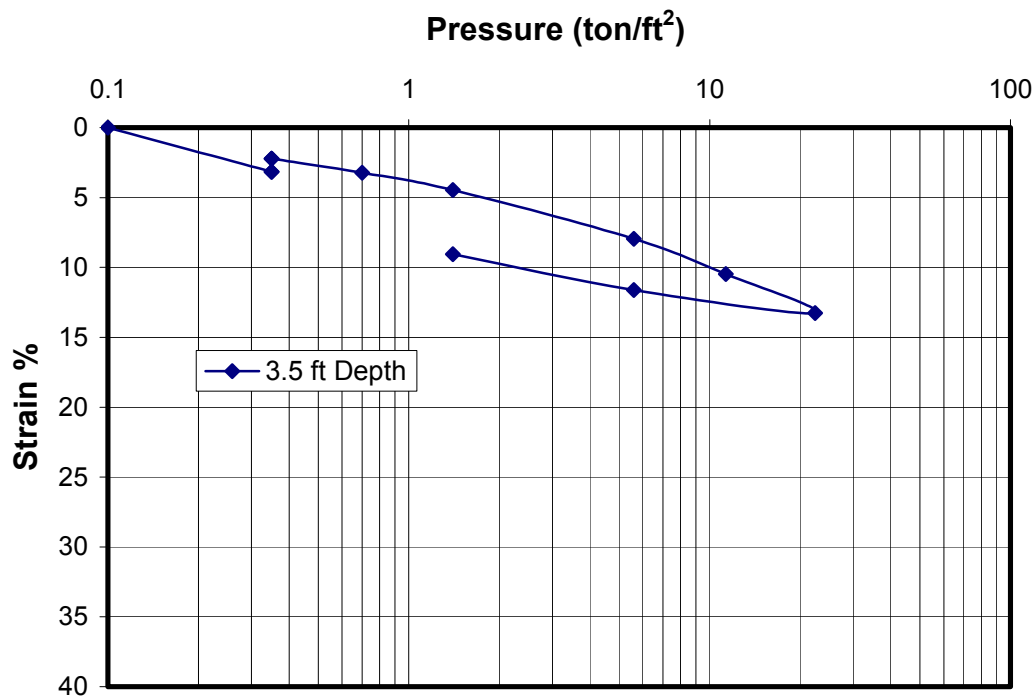


Figure 30: Percent Strain vs. applied pressure for sample at 3.5 ft in Drill Hole 1.

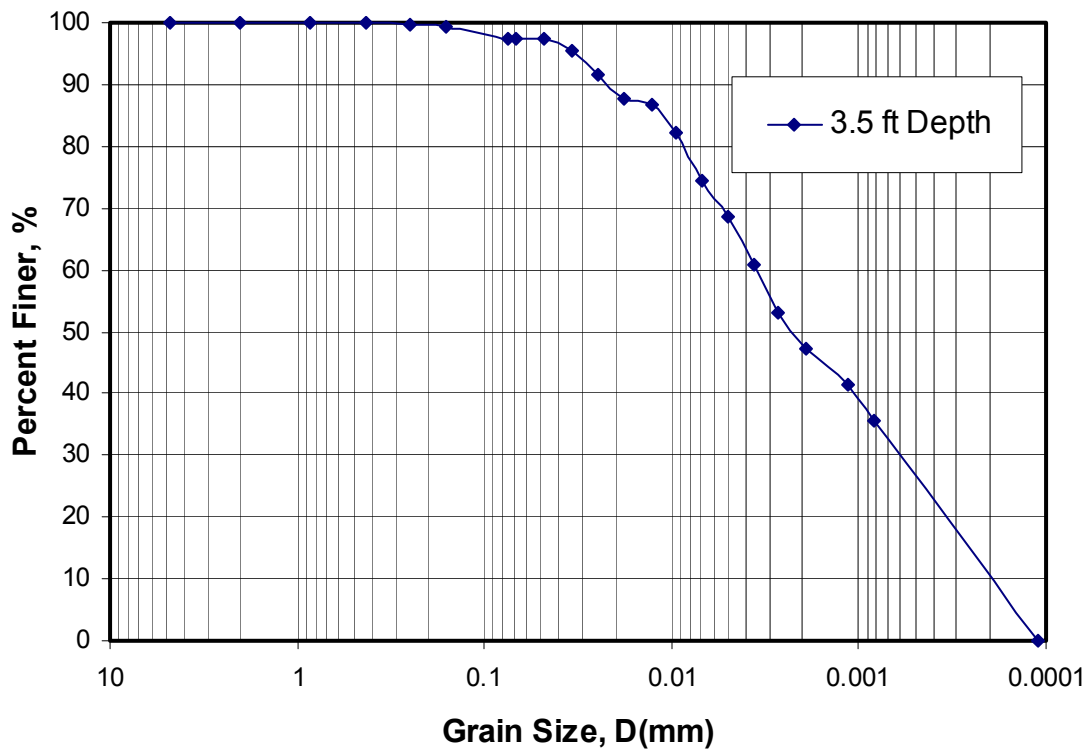


Figure 31: Grain-size distribution tests results for sample at 3.5 ft in Drill Hole 1.

Soil Conditions Near Drill Holes 2 and 5

Drill Holes 2 and 5 were grouped together because they are both located in geologic environments where collapsible soils were expected and detected. Their locations and milepost locations are shown in Figures 32 and 33. Both Drill Holes 2 and 5 were drilled through stream alluviums, Qa1. The stream alluvium cuts through the Hall Canyon Conglomerate, Trgh. A combination of collapsible soils, poor drainage, and cut/fill transition zones appear to be causing damage to the highway within the areas around Drill Holes 2 and 5. Figures 32 and 33 show these distress features and where they are located relative to Drill Holes 2 and 5.

Drill Hole 2 is located on a shallow eastward dipping slope, west of the southbound lanes of I-15 (see Figure 32). Drill Hole 2 is about 0.48 miles south of milepost 210. It was drilled to a depth of 19.5 feet through clay and sand beds underlain by sand and gravel beds. Collapsible soils were identified from the surface to at least a depth of 10 feet. The potential for collapse appears to be substantially lower in the soils below a depth of 18.5 feet.

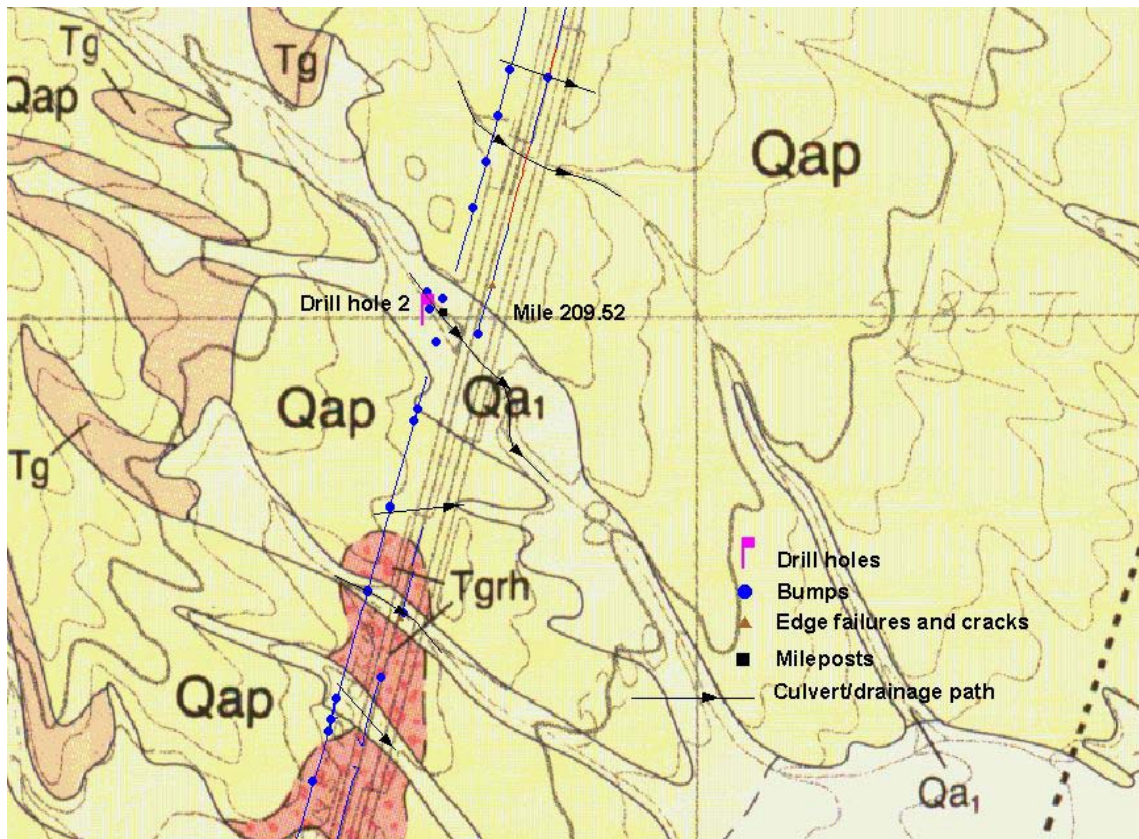


Figure 32: Drill Hole 2 and the distress features nearby, I-15, Juab County, UT. Cuts are in blue and fills are in red.

Drill Hole 5 is located south of a large cut, on a shallow eastward dipping slope, west of the southbound lanes of I-15 (see Figure 33). Drill Hole 5 is a few hundred feet north of milepost 209. This hole was drilled to a depth of 16.5 feet through clay and sand beds, sand and gravel

beds, and into a weathered wacke/tuff bed. The soils in the upper portion of Drill Hole 5 (<15 ft) were found to be collapsible, however the sample at a depth of 16 feet was expansive. Based on the test results from Drill Holes 2 and 5, other stream alluvium deposits represented by geologic unit Qa1 in Figures 32 and 33 also likely contain collapsible soils. Boring logs for Holes 2 and 5 are presented in Figures 34 and 35, respectively.

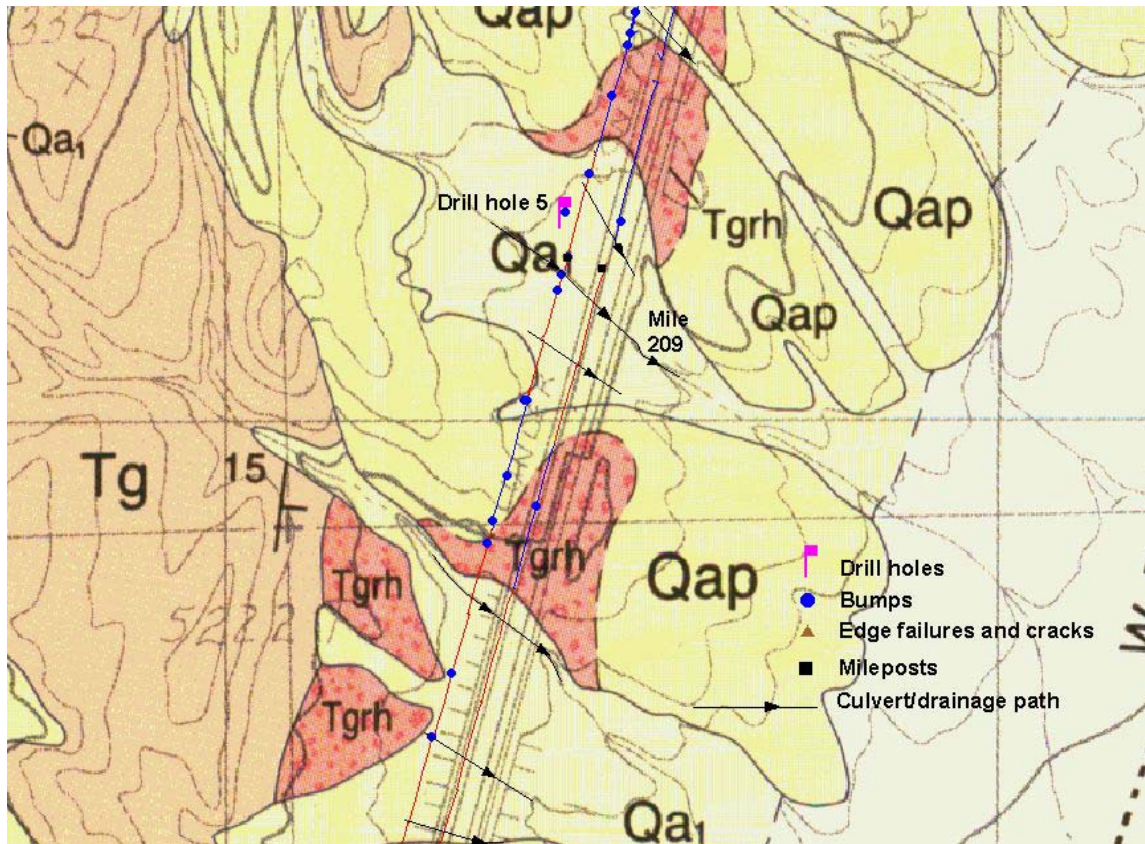















Figure 33: Drill Hole 5 and the distress features nearby, I-15, Juab County, UT. Cuts are in blue and fills are in red.

There are 33 different distress features found on the maps of the areas around Drill Holes 2 and 5 (Figures 30 and 31). Of these 33 features, 31 are bumps and 2 are edge failures. Nine bumps are found within 100 feet of a culvert/drainage path. Four bumps are found within 100 feet of a cut/fill transition zone. One edge failure and 2 bumps are found within 100 feet of both a culvert/drainage path and a cut/fill transition zone. One edge failure and 16 bumps are possibly related to just the subsurface geology. The majority of the soil samples from both holes were found to be collapsible in nature with a few expansive materials at depth.

LOG_OF_BORING_REV_1_NEPHI.GPJ_LANE_GDT_4/10/01

BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449			Boring No: BH-2 Logged By: Sizemore Date Started: 7/27/2000 Date Finished: 7/27/2000										Sheet 1 of 1 Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air			NEPHI I-15							
Location			Water Level	Depth (ft)	Graphic Log	Sample Data								% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS
Material Description						Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N ₁₆₀) Blows/ft	Soil Classification											
												USCS	AASHTO										
silty clay, brown, dry trace of gravel						2-1.5	MC	$\frac{18}{14}$	10-10-13	23	CL	A-6	13	20	68	89.9	3.9	30	17	13			
silty clay, brown, damp				5		2-5	US	$\frac{24}{24}$			CL	A-6	0	35	65	80.1	7.0	31	17	14			
silty gravel and sand, brown, dry - loose						2-7.5	MC	$\frac{18}{10.5}$	11-11-10	21	GM		45	30	25		5.3						
sandy silt, brown, very damp, traces of clay and gravel - medium density				10		2-10.5	MC	$\frac{18}{15}$	3-9-5	14	GW	A-1-b	80	15	5		3.2						
						2-14	US	$\frac{24}{9}$			SC		25	30	45		14.3						
silty sand and gravel, brown, damp, traces of clay - loose				15		2-16.5	MC	$\frac{18}{12}$	11-14-12	26	GC	A-2-4	43	22	29		11.5	26	16	10			
silty fine sand, brown, very damp				20		2-19.5	MC	$\frac{18}{10.5}$	6-8-6	14	GW	A-1-b	80	10	10	102.4	12.9						
LEGEND AND NOTES			 SPT = Standard Penetration Test, Split Spoon Sample  US = Undisturbed Shelby Tube, Pushed  Ring = Dames & Moore Sampler  Bulk = Bulk Sample From Cuttings								GROUNDWATER DEPTHS:  = Initial  = End of Drilling			COMMENTS: Fig. 34 Boring log and test results for Drill Hole 2									

MC=Modified California Sampler

BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449			Boring No: BH-5 Logged By: Sizemore Date Started: 7/27/2000 Date Finished: 7/27/2000			Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air			Sheet 1 of 1			NEPHI I-15										
Location			Water Level	Depth (ft)	Graphic Log	Sample Data						% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS	
						Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N _{1,60}) Blows/ft						Soil Classification					
Material Description																	USCS	AASHTO				
silty clay, brown, dry						5-1.5	MC	$\frac{18}{15}$	5-11-13	24	CL	A-6	0	1	90	87.3	7.0	40	21	19		
						5-4.5	MC	$\frac{18}{15}$	8-9-9	18	CL	A-6	0	13	86	81.6	5.7	47	20	27		
silty sand with gravel, brown, dry				5																		
silty sand to silty fine sand, brown, dry						5-7.5	MC	$\frac{18}{15}$	5-6-12	18	SP-SM		1	22	77	82.4	12.7					
silty sand with some gravel, damp, brown-grey - medium dense						5-10.5	MC	$\frac{18}{15}$	16-12-10	22	SP					90.5	9.8					
						5-13.5	MC	$\frac{18}{11}$	18-19-25	44	SP					7.9	39	16	23			
						5-16.5	MC	$\frac{18}{16}$	16-17-31	48	SC	A-7-6	0	53	47	105.3	15.6	59	25	34		
grey silty sandstone, damp to very damp																						
grey shale or claystone, damp																						
</																						

Figures 36 through 41 are pictures of areas and distress features taken around Drill Hole 2. They show where Drill Hole 2 is located, troughs and bumps around Drill Hole 2, and a plant-filled drainage ditch leading to a culvert that passes from west to east under I-15. Troughs and bumps are near the culvert and might be related to soils collapsing around the culvert when water permeates into the soil. Figures 42 through 45 are pictures of areas and distress features around Drill Hole 5. They show where Drill Hole 5 is located, bumps along the highway, and a drainage ditch leading to a culvert that passes from west to east under I-15. Bumps are also near this culvert and might be related to soils collapsing around the culvert when water permeates into the soil (Fig. 45).



Figure 36: Looking west at the area around Drill Hole 2 near the southbound lanes of I-15. Drill Hole 2 was drilled in the center of the photo through the stream alluvial deposits creating the flat section between the alluvial fans.



Figure 37: Trough causing loss of ride quality. The trough is near a drainage and near Drill Hole 2, Juab County, UT.

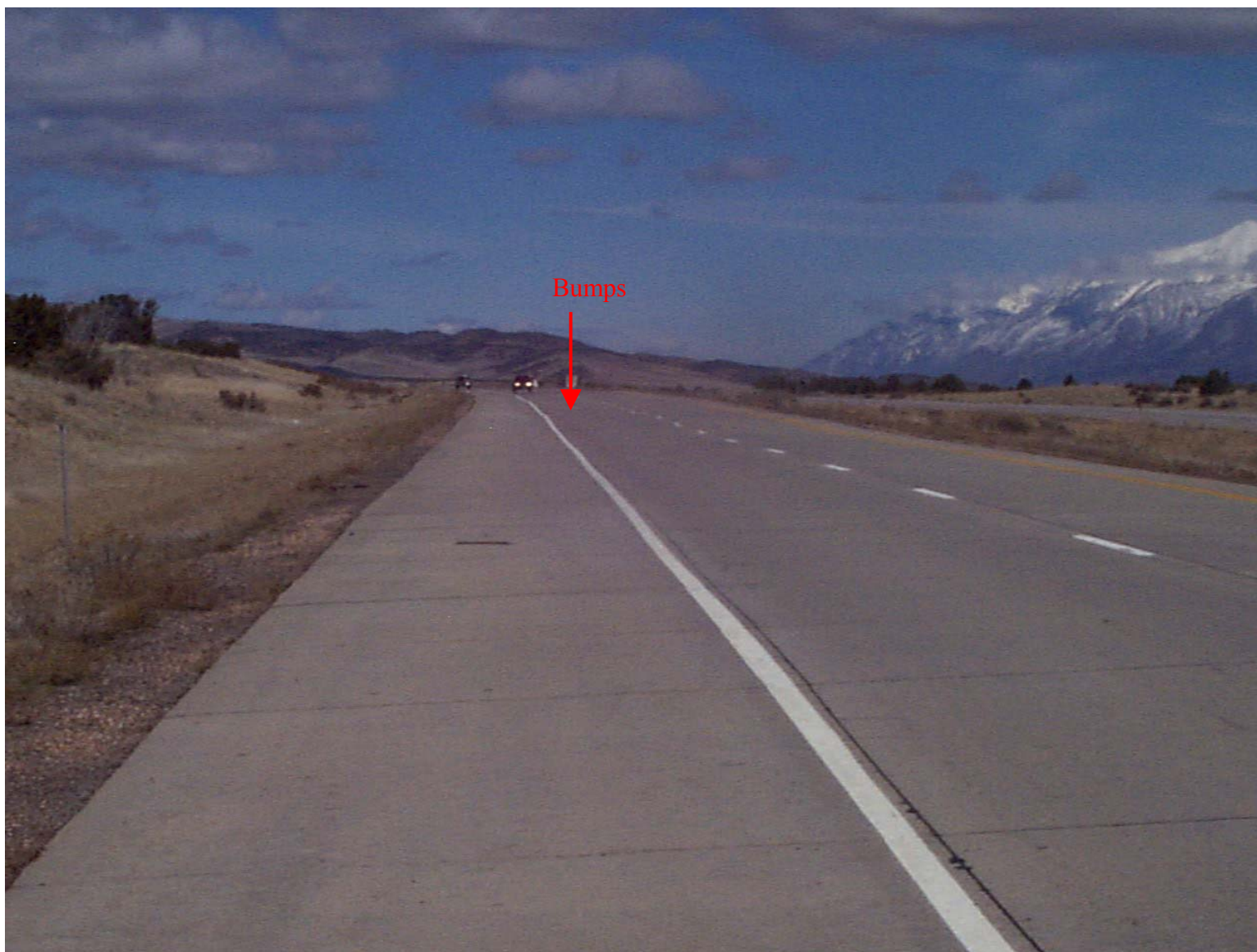


Figure 38: Looking north from Drill Hole 2 along the southbound lanes of I-15. Several bumps can be seen in the middle of the picture.



Figure 39: Looking south from Drill Hole 2 along the southbound lanes of I-15. A trough and several bumps can be seen in the middle of the picture.



Figure 40: Plant growth at drainage near Drill Hole 2, Juab County, UT.



Figure 41: Looking west at the area around Drill Hole 5 near the southbound lanes of I-15. Drill Hole 5 was drilled in the center of the photo through the stream alluvial deposits creating the flat section between the alluvial fans.



Figure 42: Elevated view of the area around Drill Hole 5, looking southeast. I-15 goes through the center of the picture. A bump can be seen in the repaired section on the freeway.



Figure 43: A significant bump can be seen on the northbound lanes of I-15 in the middle of the picture. This picture was taken looking north and Drill Hole 5 is to the west (not in picture).



Figure 44: Looking south from Drill Hole 5 along the southbound lanes of I-15. Several bumps can be seen in the middle of the photo.



Figure 45: One of the two culverts that passes under I-15 from west to east near Drill Hole 5. The drainage is partially blocked by plant growth.

The samples from Drill Holes 2 and 5 were tested in the BYU Soil Mechanics laboratory to determine Atterberg limits, grain-size distribution, and soil classification. In addition, consolidation tests were conducted to determine the collapse or swell potential after the samples were wetted following loading to the overburden pressure. The testing showed that collapsible soils do exist in Drill Holes 2 and 5. Expansive soils also exist at deeper levels in Drill Hole 5. The Atterberg limits, unit weight and water content data from the lab testing are shown on the boring logs for each hole. In addition, the soil classification system symbols based on both the Unified Soil Classification System and the AASHTO classification system are shown on the logs.

The strain versus applied pressure curves from consolidation tests on five samples from Drill Hole 2 are shown in Figure 46. The increase in strain at a constant pressure when water is added indicates the potential for collapse. Figure 47 shows the collapse strain upon wetting as a function of depth below the ground surface. The maximum percent collapse in Drill Hole 2

was 10.1 % of the initial volume at a depth of 3.5 feet; however, most of the profile in the upper 10 ft experienced 3 to 4 % collapse strain upon wetting. No collapse was observed in the sample at 19.5 ft depth. Collapse tests could not be performed on samples between 10 and 19.5 ft because of the presence of gravel particles. However, based on case histories of collapsible gravels (Rollins et al. 1994) these material would likely exhibit moderate collapse potential as well. Figures 48 and 49 present grain-size distribution curves for samples at 0.5, 1, 3.5, 4.5, 10, 15.5, and 16 ft.

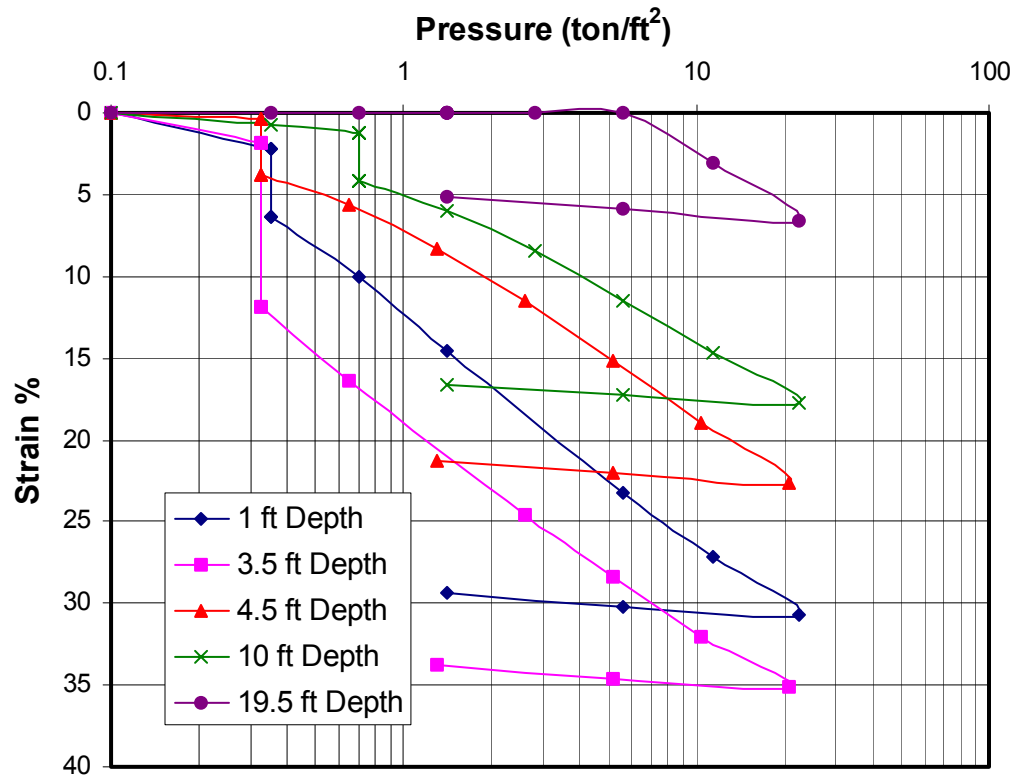


Figure 46: The percent strain as a function of applied pressure for five samples in Drill Hole 2. The change in strain at a constant pressure indicates the collapse strain due to wetting.

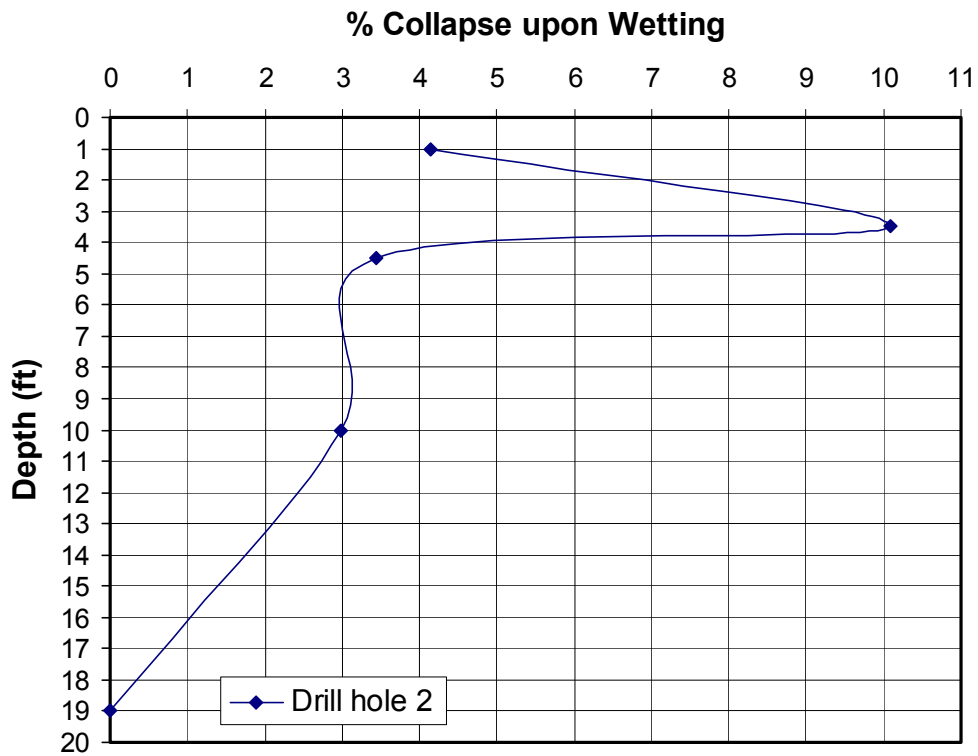


Figure 47: Percent collapse as a function of depth for Drill Hole 2.

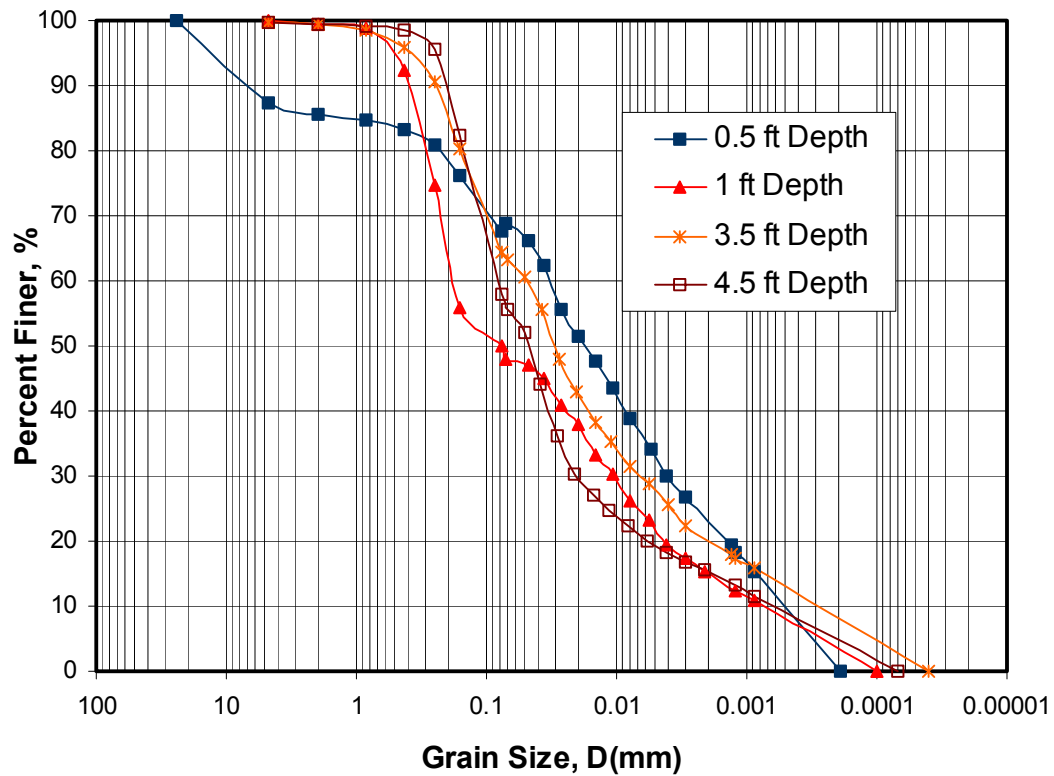


Figure 48: Grain-size distribution curves for samples at 0.5, 1, 3.5, and 4.5 ft depth in Drill Hole 2.

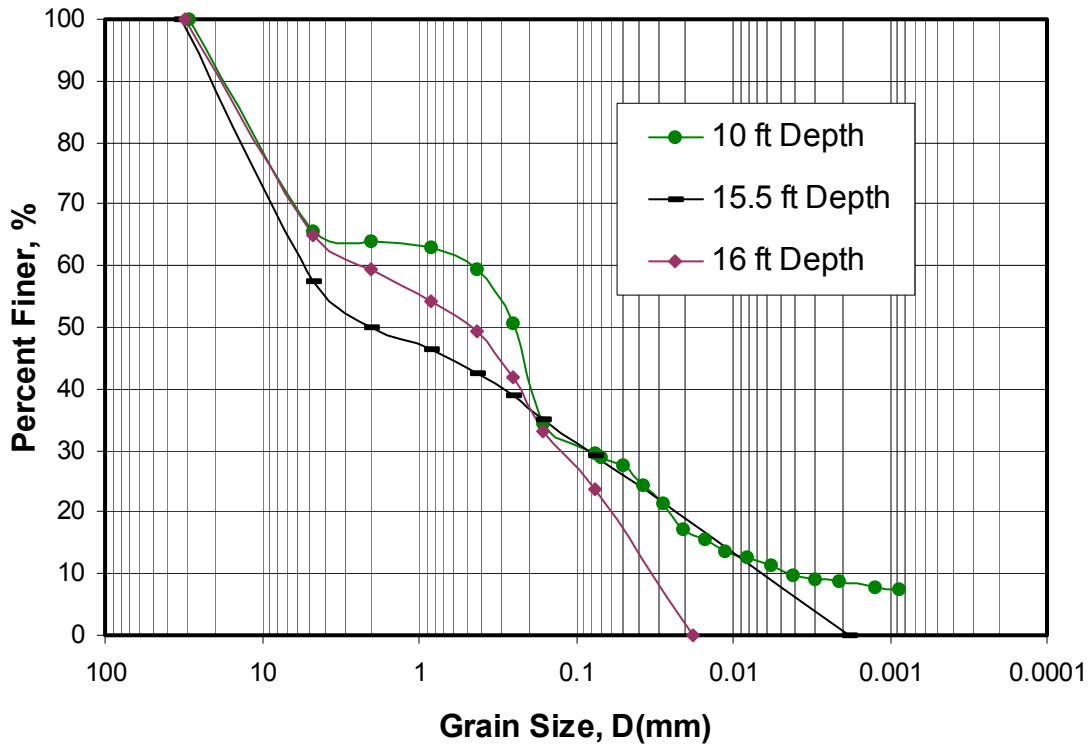


Figure 49: Grain-size distribution curves for samples at 10.5, 15.5, and 16 ft depth in Drill Hole 2.

The grain-size distribution curves indicate that the materials contain 30 to 70 % fines with up to 10 to 20 % clay size materials. While some of the materials below 10 ft contain significant gravel components, they have sufficient fines to produce a collapsible structure.

Samples at 1, 3.5, 4.5, 7, 7.5, 10, and 16 ft depths in Drill Hole 5 were also tested in the consolidometer to evaluate their potential for swell or collapse after wetting. The percent strain as a function of applied pressure is shown for each test specimen in Figure 50. The percent strain upon wetting as a function of depth is shown in Figure 51. The maximum percent collapse for Drill Hole 5 was 8.9 % of the initial height at a depth of 7 feet; however, collapse strains between 4 and 7 % were more typical. The specimen at a depth of 16 feet swelled 2.2% when water was added at the natural moisture content. Figures 52 through 54 present grain-size distribution curves for samples at various depths in Drill Hole 5. These materials typically contain 70 to 90 % fines and have clay contents of 25 to 40 %.

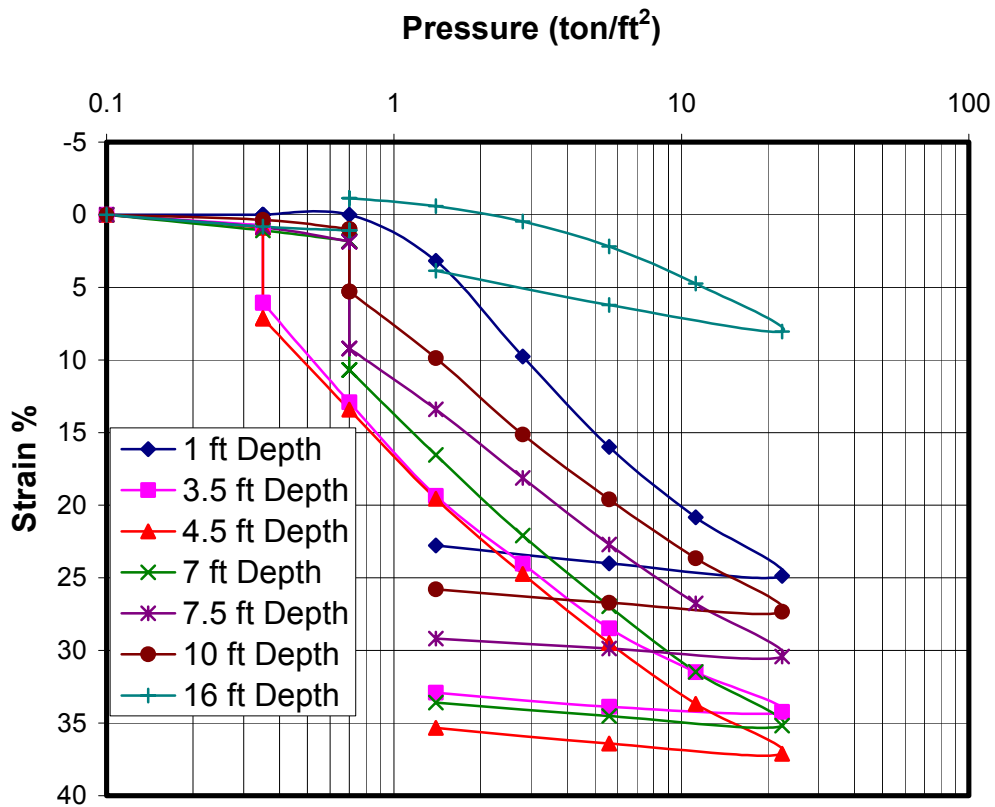


Figure 50: Percent strain as a function of applied pressure for samples at 1, 3.5, 4.5, 7, 7.5, 10, and 16 ft depth in Drill Hole 5. Increase in strain at a constant pressure following wetting indicates potential for collapse.

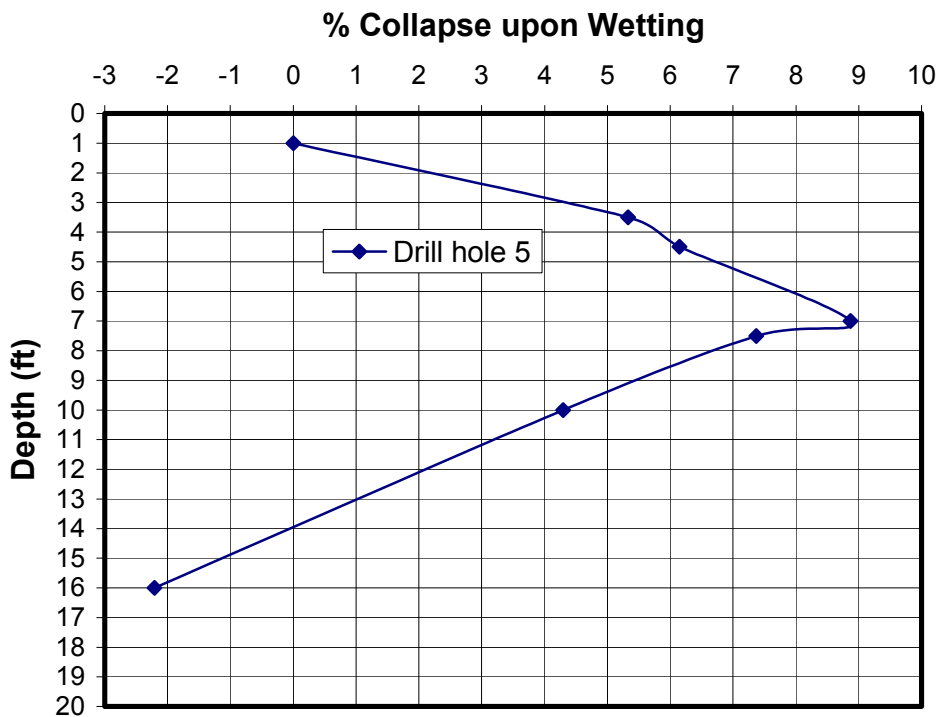


Figure 51: Percent collapse or expansive strain versus depth for samples from Drill Hole 5.

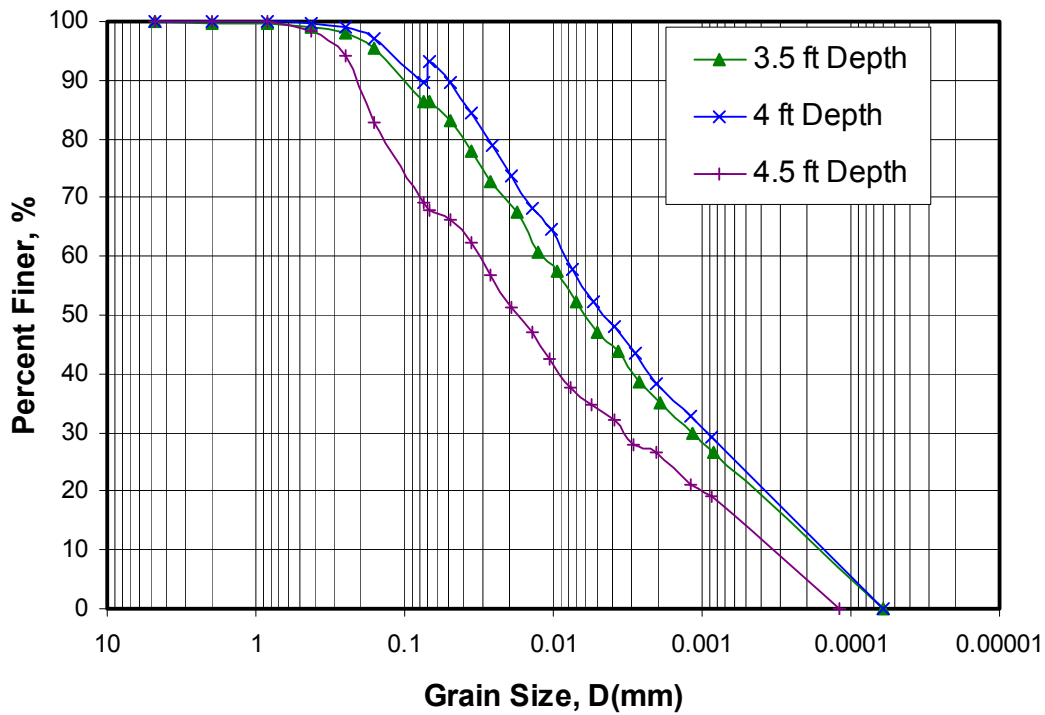


Figure 52: Results of grain-size distribution tests for samples 3.5, 4 and 4.5 ft depth in Drill Hole 5.

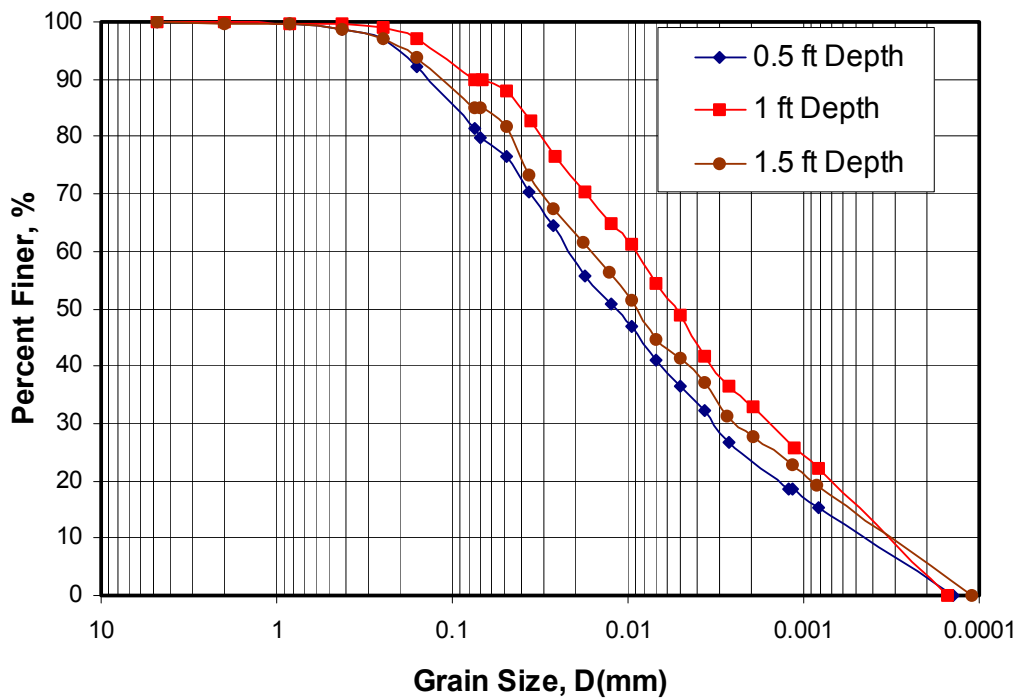


Figure 53: Results of grain-size distribution tests for samples at 0.5, 1 and 1.5 ft in Drill Hole 5

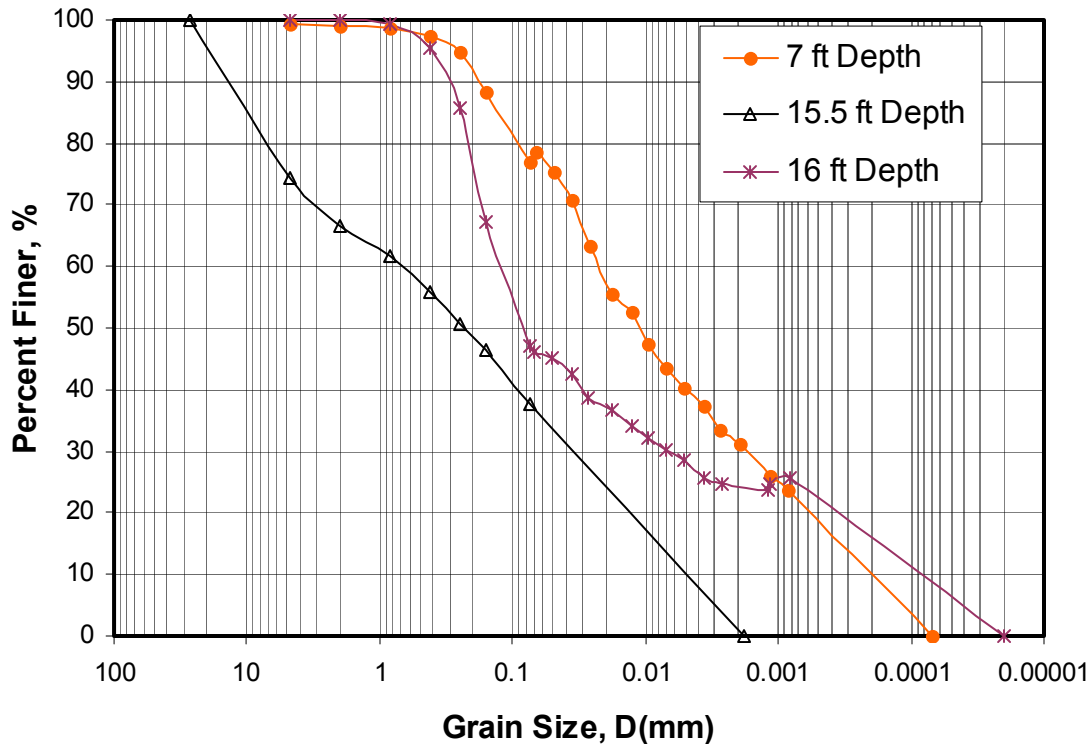


Figure 54: Results of grain-size distribution tests for samples at 7, 15.5 and 16 ft depth in Drill Hole 5.

Soil Conditions Near Drill Holes 6, 7, 8 and UDOT Drill Holes B

Drill Holes 6, 7, 8 and UDOT Drill Holes B are grouped together because they penetrate profiles where expansive soils are expected based on the geologic mapping. These holes are located between mileposts 208 and 207. Figure 55 shows the locations of Drill Holes 6, 7, and 8 along I-15, the distress features near the Drill Holes, the cut and fill sections along the highway, and the locations of the culvert and drainage paths. The distress features consist of both bumps and edge failures and most of them occur within the large cut section between holes 6 and 8. A combination of expansive soils found within the cut section and poor drainage running parallel to the highway are the likely cause for most of the problems in this area.

Drill Hole 6 was located within a fill section between the southbound and northbound lanes of traffic. It is also located at the south end of the west cut section and about 0.7 miles south of milepost 208. The overburden consisted of layers of weathered siltstone fill underlain by fine gravel in a matrix of weathered siltstone. Claystone/mudstone (Tg) was encountered at a depth of 3.5 ft. The claystone was too hard to penetrate with a sampler; however, hand cut samples of the weathered claystone were obtained from the base of the cut section near Drill Hole 6 and were found to be expansive. Drill Hole 7 was located in a fill section between the northbound and southbound lanes of traffic. It is about 0.2 miles south of milepost 208 and near the north end of the east cut section. The hole was drilled to a depth of 6.8 feet through a thin layer of silty sand with gravel (1.5 ft thick) and into weathered shale/mudstone (Tvg).

Drill Hole 8 is located at the base of the east cut section about 100 feet north of Drill Hole 7. It was drilled to a depth of 10.5 feet through a thin layer of clay and gravel and into a weathered shale/mudstone (Tvg). A few hand cut samples were also taken from the area around Drill Hole 8. Most of the soils from Drill Hole 8 were found to be expansive; however, the soils in the upper foot of the profile showed collapse characteristics. Boring logs for Drill Holes 6, 7 and 8 are shown in Figures 56, 57, and 58, respectively. UDOT Drill Holes B were located about 0.4 miles north of milepost 207. Boring logs for the four holes are shown in Figure 59.

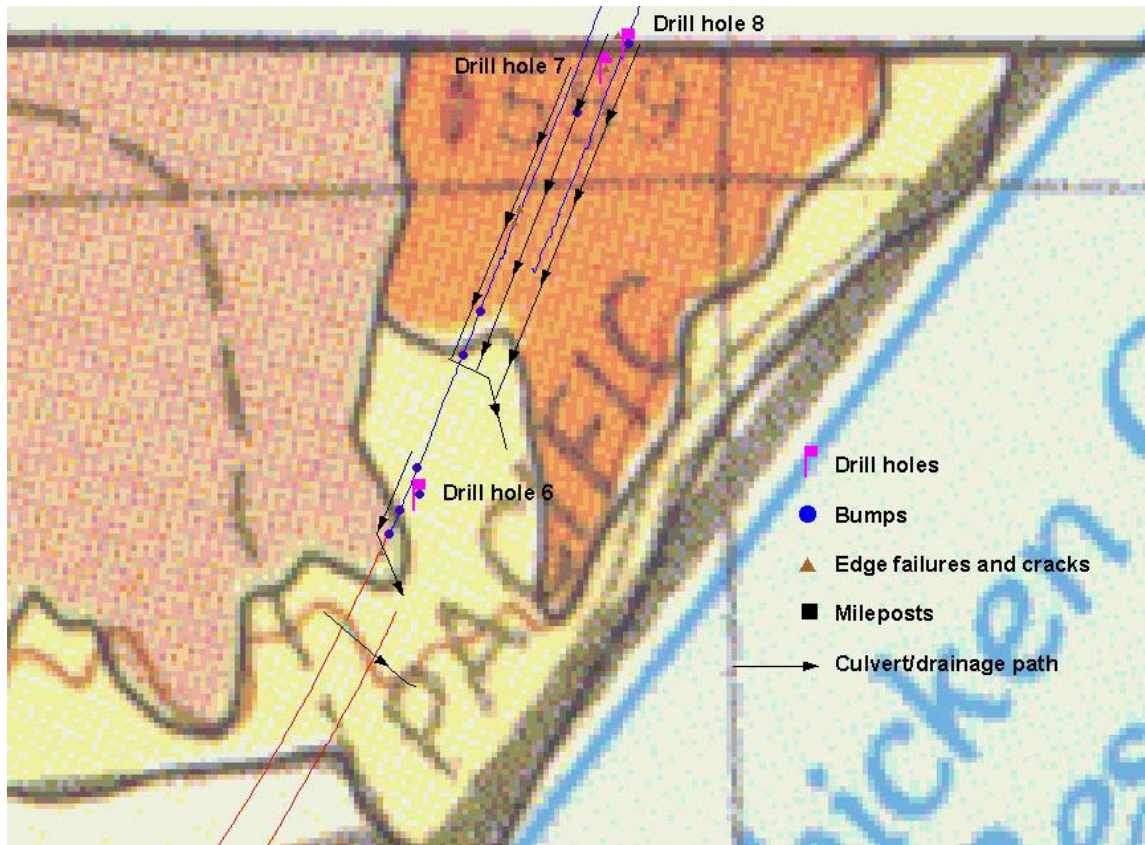

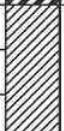
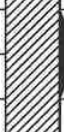

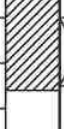
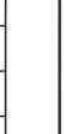










Figure 55: Drill Holes 6, 7, and 8 and the distress features nearby, I-15, Juab County, UT. Cuts are in blue and fills are in red.

There are 13 distress features found on the map of the area around Drill Holes 6, 7 and 8 (see Fig. 55). Seven of the distress features are edge failures and six are bumps. Seven edge failures and five bumps are found within 100 feet of a culvert/drainage path. One bump is within 100 feet of both a culvert/drainage path and a cut/fill transition. All of the distress features appear to be related to expansive soils which swell and shrink with changes in the moisture content. The drainage ditches along the highway are allowing water to pond. This water can then permeate into the soil and lead to heaving of the highway. In addition, when the water reaches the shales, the potential for weathering is increased and even greater problems with expansion can occur. This phenomenon was clearly evident at locations where hand samples were cut. The shale had weathered and softened into expansive clay in zones where water had

BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449		Boring No: BH-6 Logged By: Sizemore Date Started: 8/1/2000 Date Finished: 8/1/2000		Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air		Sheet 1 of 1 NEPHI I-15														
Location	Water Level	Depth (ft)	Graphic Log	Sample Data					% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS		
Material Description			Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N ₁) ₆₀ Blows/ft	Soil Classification							LL			PL	PI
									USCS	AASHTO										
broken siltstone probably fill, some sand and gravel highly weathered siltstone with some claystone, grey, some sand and fine gravel gravel, mostly fine in a siltstone matrix, grey claystone with some sand and a trace of gravel, grey, too hard to penetrate with sample, a few thin beds of dark grey claystone		1 2 3 4 5 6 7 8 9 10		6-1	SPT	12 12	30-50	refusa	CL	A-6					7.3	49	18	31		
LEGEND AND NOTES SPT = Standard Penetration Test, Split Spoon Sample US = Undisturbed Shelby Tube, Pushed Ring = Dames & Moore Sampler Bulk = Bulk Sample From Cuttings				GROUNDWATER DEPTHS: = Initial = End of Drilling				COMMENTS: Fig. 56 Boring log and test results for Drill Hole 67.												

BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449			Boring No: BH-7 Logged By: Sizemore Date Started: 8/2/2000 Date Finished: 8/2/2000			Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air			Sheet 1 of 1			NEPHI I-15											
Location			Water Level	Depth (ft)	Graphic Log	Sample Data						% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS		
Material Description						Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N _{1,60}) Blows/ft						Soil Classification		Liquid Limit			Plastic Limit	Plasticity Index
																	USCS	AASHTO					
silty sand, some gravel, brown, fill, dry - medium dense				1		7-1.5	MC	18 15	7-7-12	19	CH	A-7-6	2	13	85		11.8	69	19	50			
clayey siltstone, highly weathered, brown, damp				2																			
				3																			
				4		7-4	MC	12 11	31-63	refusa	CL	A-7-6	0	0	100	123.0	9.1	41	20	21			
				5																			
				6																			
				7		7-6.8	SPT	9 9	30-40	refusa	CL	A-7-6				15.1	43	19	24				
				8																			
				9																			
				10																			
			MC=Modified California Sampler																				
LEGEND AND NOTES			 SPT = Standard Penetration Test, Split Spoon Sample  US = Undisturbed Shelby Tube, Pushed  Ring = Dames & Moore Sampler  Bulk = Bulk Sample From Cuttings			GROUNDWATER DEPTHS:  = Initial  = End of Drilling			COMMENTS: Fig. 57 Boring log and test results for Drill Hole 7.														

LOG OF BORING REV. 1 NEPHI.GPJ LANE.GDT 4/10/01

BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449			Boring No: BH-8 Logged By: Sizemore Date Started: 8/2/2000 Date Finished: 8/2/2000		Sheet 1 of 1 Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air		NEPHI I-15														
Location			Water Level	Depth (ft)	Graphic Log	Sample Data						% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS
Material Description						Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N _{1,60}) Blows/ft						Soil Classification				
																	USCS	AASHTO			
silty sand and gravel, fill																					
sandy silt with some clay and gravel, brown, dry																					
silty claystone, brown and some grey, damp, weathered to highly weathered				5		8-4.3	MC	$\frac{16}{12}$	19-50-70	refusa	CH	A-7-6	1	5	95	126.5	12.7	54	18	36	
highly weathered to weathered siltstone, damp				10		8-6.8	SPT	$\frac{9}{9}$	22-50	refusa	CH	A-7-6				12.7	56	18	38		

LOG OF BORING, REV. 1 NEPHI.GPJ LANE GDT 4/10/01

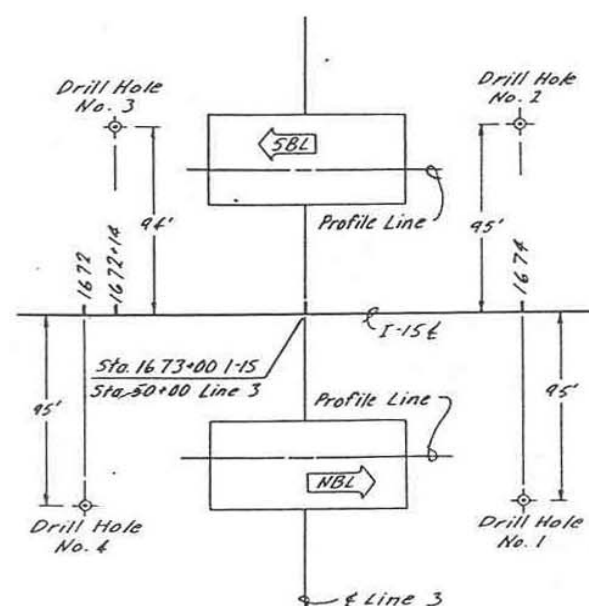
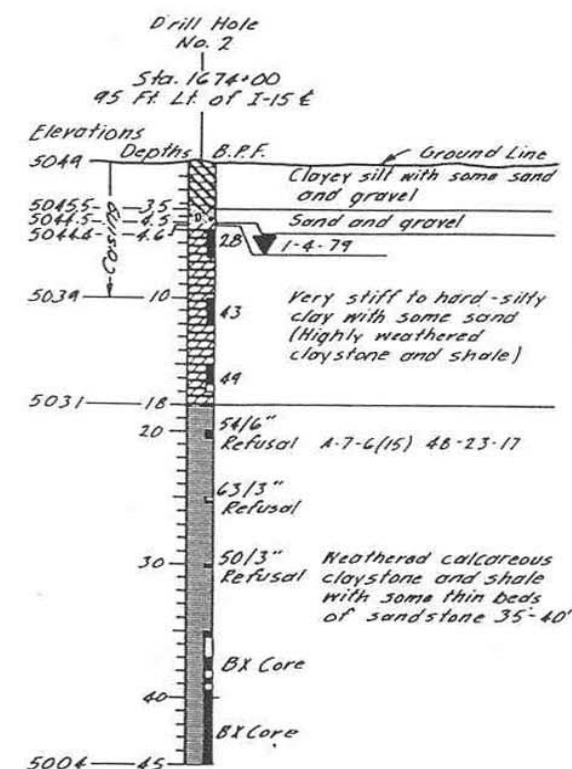
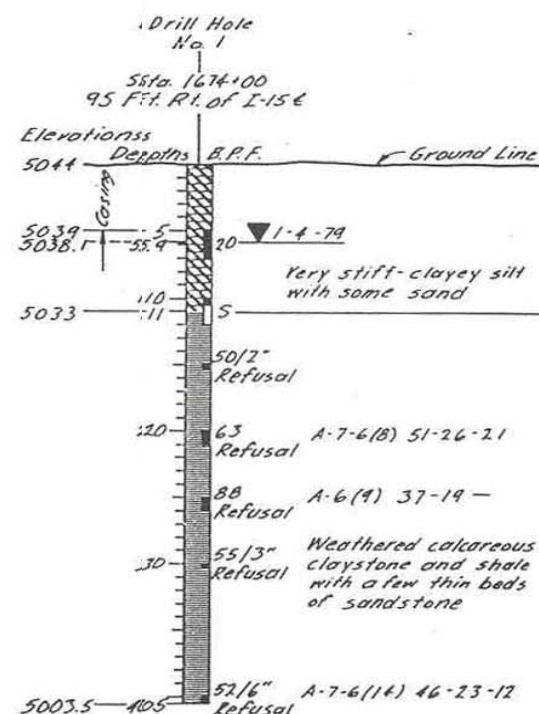
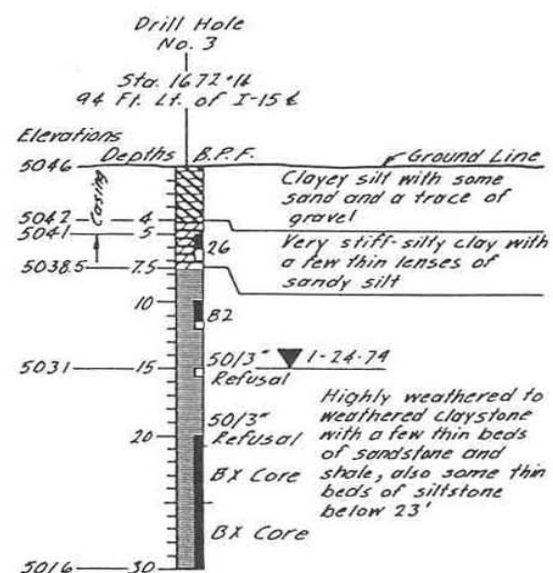
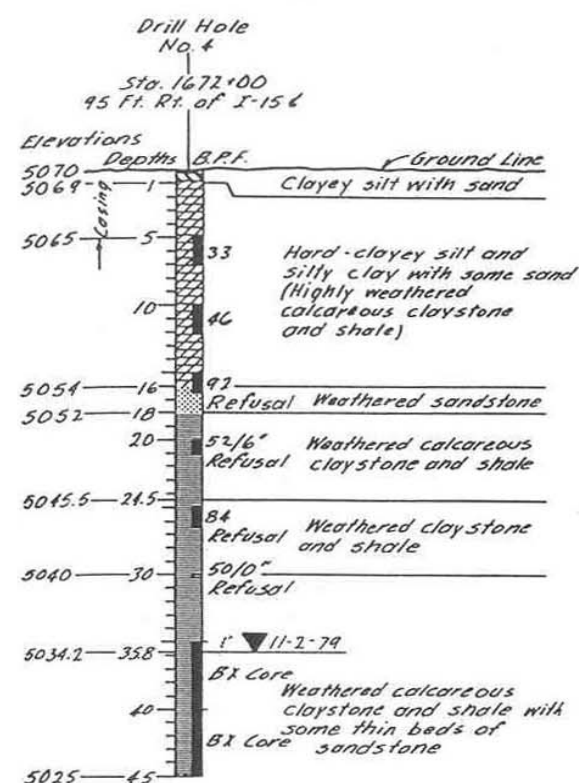


Figure 59: Boring logs for UDOT Drill Holes B.

KEY TO DRILLING LOG

RELATIVE DENSITY (NON-PLASTIC SAND & SILT)

VERY LOOSE - LESS THAN 4 BLOWS PER FOOT.

LOOSE - 4 TO 10 BLOWS PER FOOT.

MEDIUM - 10 TO 30 BLOWS PER FOOT.

DENSE - 30 TO 50 BLOWS PER FOOT.

VERY DENSE - MORE THAN 50 BLOWS PER FOOT.

CONSISTENCY (PLASTIC SILT & CLAY)

VERY SOFT - LESS THAN 2 BLOWS PER FOOT.

SOFT - 2 TO 4 BLOWS PER FOOT.

MEDIUM - 4 TO 8 BLOWS PER FOOT.

STIFF - 8 TO 15 BLOWS PER FOOT.

VERY STIFF - 15 TO 30 BLOWS PER FOOT.

HARD - MORE THAN 30 BLOWS PER FOOT.

TOPSOIL OR FILL

GRAVEL

SAND

SILT

CLAY

SHALE

IGNEOUS

LIMESTONE

CONGLOMERATE

DOLomite

SANDY CLAY

CLAYEY SAND

SILTY CLAY

CLAYEY SILT

SILTY SAND

SANDY SILT

DRILL HOLE NO. 0+00 E OR L OR RE IN FT. OFFSET.

STATION

ELEVATIONS

GROUND ELEVATION

DEPTHS

GROUND LINE

EXAMPLE TYPICAL STIFF MEDIUM PLASTIC BRN. CLAY, SOME SILT

AASNO LL-PI-W IT-7-11

4555

DATE

GROUND WATER TABLE

4552

STRATA CHANGE

4546

LOCATION OF SAMPLE

20'

25'

30'

THIN WALL SHELBY TUBE, UNDISTURBED SAMPLER USED.

SPLIT BARREL UNDISTURBED SAMPLER WITH LINER RINGS OR CALIFORNIA TYPE SAMPLER

SAMPLE NOT RECOVERED

REASON NOT RECOVERED

NO. OF BLOWS OF A 140 LB. HAMMER FALLING 30 INCHES REQUIRED TO DRIVE A STD. 1 1/2" O.D. SAMPLE TUBE 1 FT.

CLASSIFICATION OF EACH SAMPLE AND RESULTS OF CLASSIFICATION TESTS.

ABBREVIATIONS

L.L. - LIQUID LIMIT IN %

P.I. - PLASTIC INDEX

W. - NATURAL MOISTURE CONTENT IN %

W.G. - WELL GRADED

PEN. - PENETRATION

G.W.T. - GROUND WATER TABLE

B.P.F. - BLOWS PER FOOT.

N.P. - NON PLASTIC

NOTE: REFUSAL = 50 or more blows per 6"

UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
MATERIALS AND RESEARCH SECTION

MILLS JUNCTION INTERCHANGE

SOIL DATA

DATE BY: D. Kistler CHECKED BY: H.C. Smith I-15-5(22)207

CHECKED BY: M. Kistler CHECKED BY: S. Kisthal

DATE: 5-21-84 DATE: 5-21-84

STATION: 1673+00

QUANTITY: JUAB

BR. NO. F-435 2 of 11

Date Drilled: 1-79 and 10-79

NO.	BY	DATE	REVISIONS

ponded but was still intact a few feet away from the zone of wetting. This process can also lead to significant differential movement of the pavement.

Figures 60 through 64 are pictures of the area and distress features around Drill Hole 6. They show where Drill Hole 6 is located, a surface overlay near Drill Hole 6, bumps, pavement offsets, and a plant-filled drainage along the west cut bank of I-15. Figures 65 through 69 are pictures of the area and distress features around Drill Holes 7 and 8. The pictures show where Drill Holes 7 and 8 are located near the northbound lanes of I-15, longitudinal cracks making up part of the edge failures within this study, a large bump, and a rock-filled drainage ditch along the east cut bank that has ponded water in it. A similar situation was observed for the drainage ditch running down the median in this section of the freeway.



Figure 60: Elevated view of the area around Drill Hole 6, looking southeast. I-15 goes through the middle of the picture. Drill Hole 6 was drilled in the middle of the picture through the east fill slope of the southbound lane.



Figure 61: Bump and surface overlay along the southbound lane of I-15. Picture was taken looking north and Drill Hole 6 is to the east (not shown).



Figure 62: Pavement offset from a bump along I-15 southbound, near Drill Hole 6, Juab County, UT.



Figure 63: A second pavement offset along I-15 southbound, near Drill Hole 6, Juab County, UT.



Figure 64: Plant-filled drainage on the western side of the southbound lane of I-15 near Drill Hole 6. The ground is very soft due to ponding of water and percolation into the subgrade.



Figure 65: Looking northwest at the area around Drill Holes 7 and 8. Drill Hole 7 was drilled through the west fill bank of the northbound lanes of I-15. Drill Hole 8 was drilled through the east cut section east of the northbound lanes of I-15. (Large longitudinal crack extending from the right side of the picture to the left side of the picture is shown in red.)

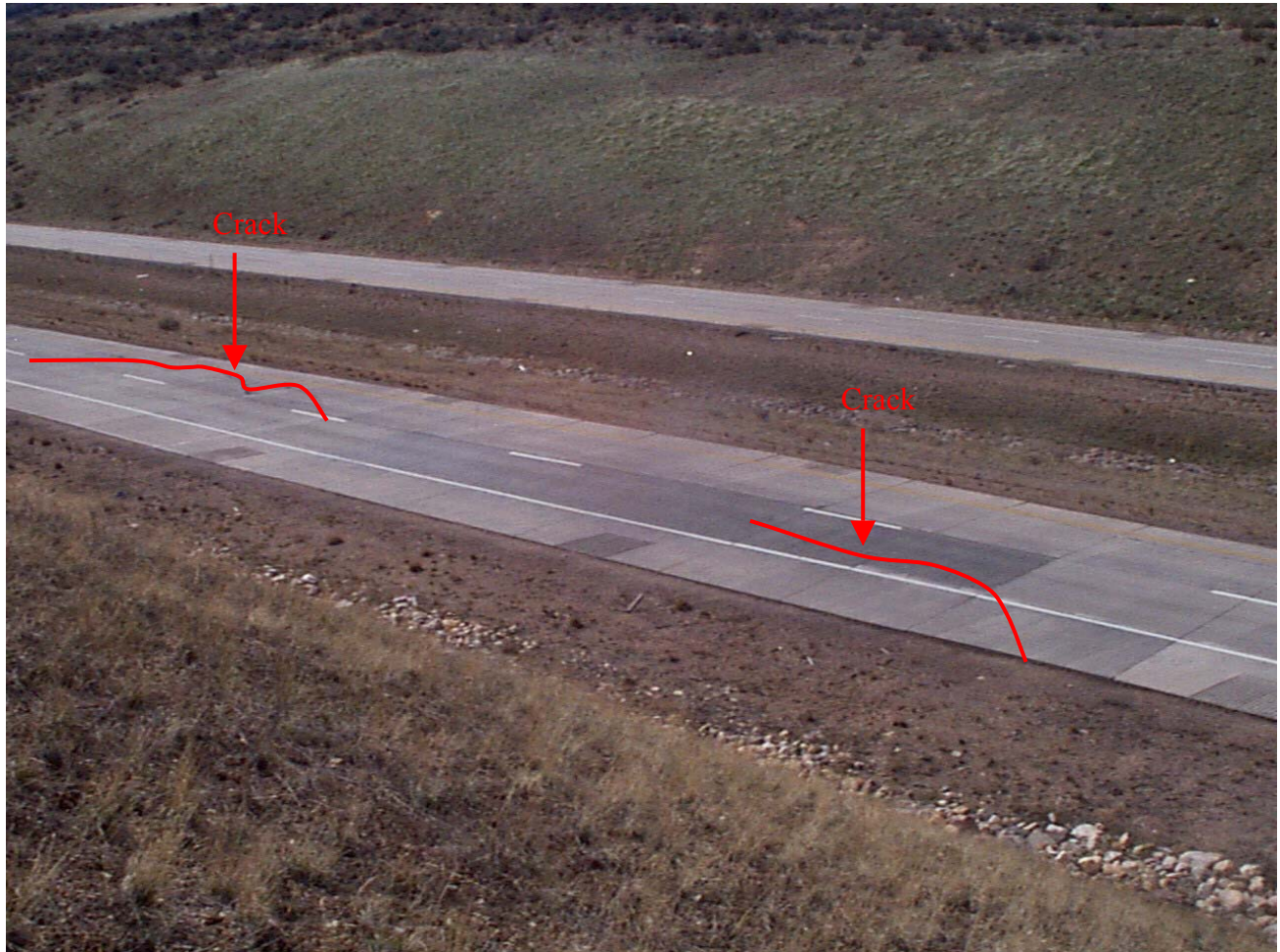


Figure 66: Looking southwest at the area around Drill Holes 7 and 8. There are two larger cracks along the northbound lanes of I-15 seen in this picture. The first is located in the upper $\frac{1}{4}$ of the lower right $\frac{1}{2}$ of the picture. The second is located in the lower $\frac{1}{4}$ of the upper left half of the picture.



Figure 67: Cracking along the northbound lane of I-15 near Drill Holes 7 and 8. Close-up of Figure 66.



Figure 68: Large bump creating a danger for drivers spanning across the northbound lanes of I-15. This picture was taken looking north and is near Drill Holes 7 and 8.



Figure 69: Water ponding in drainage ditch along the east cut section of I-15 near Drill Holes 7 and 8.

The samples from Drill Holes 6, 7, and 8 were taken back to the laboratory at BYU and tested. Tests were performed to determine the Atterberg limits, the grain-size distribution, and the collapse or expansion of the soils following wetting. The results of the grain-size distribution and Atterberg limit testing are shown on the boring logs and indicate that the soils are nearly all fine-grained materials with relatively high plasticity. The hydrometer analysis shows the weathered shale typically contained 20 to 40 % clay size particles. The plasticity index ranged from 21 to 50 for the samples from the drill holes. Atterberg limits were also determined for four samples of the clay close to the ground surface at locations between Drill Holes 6 and 8 and the results are summarized in Table 1. Once again, the plasticity index values are above 50 in three of the four cases. These high plasticity index values strongly suggest that these soils derived from weathered tuffs, claystones and shales in this region will be expansive. The soil classifications according to both the Unified Soil Classification System and the AASHTO classification systems are shown on the boring logs and in Table 1. The soils typically classified as A-7-6 soils using the AASHTO system and CH soils using the USC system

Table 1: Atterberg limit test results from hand-cut samples between Drill holes 6 and 8.

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	Classification
2 @ 2 ft depth	52	24	28	A-7-6 (CH)
4 @ 1 ft depth	88	25	63	A-7-6 (CH)
6 @ 2 ft depth	82	31	51	A-7-6 (CH)
13@ 1 ft depth	85	25	60	A-7-6 (CH)

The measured strain versus applied pressure curves from consolidation tests on samples from Drill Hole 6 are shown in Figure 70. The samples were initially loaded at the natural moisture content and then water was added to saturate the samples when the pressure reached the overburden pressure. The percent swell as a function of depth is shown in Figure 71. The maximum swell percentage following wetting was only 1.5 %; however, it should be recognized that the natural moisture content of the samples was relatively high prior to testing. Substantial swell likely would have already occurred prior to the laboratory testing.

Perhaps a better indication of the change in volume due to moisture fluctuation is provided by the shrinkage observed when the samples were dried subsequent to testing. For example the specimens from Drill Hole 6 experienced a volumetric strain of 15 to 25 % due to drying. This shrinkage due to drying is illustrated in Figure 72 where the test rings are shown along with the soil specimens following drying. The soil specimens initially had the same diameter as the steel ring. A sample of collapsible soil with a low plasticity index has a diameter following drying which is slightly smaller than the test ring; however, the sample of expansive soil has a diameter which is much smaller than the test ring due to shrinkage. A close-up view of the dried expansive soil sample relative to the test ring is shown in Figure 73. This potential for shrinkage and swelling when the water content fluctuates with the seasons can lead to cracking and differential movement of a pavement structure. This process will continue unabated over time unless the expansive soil is removed or treated.

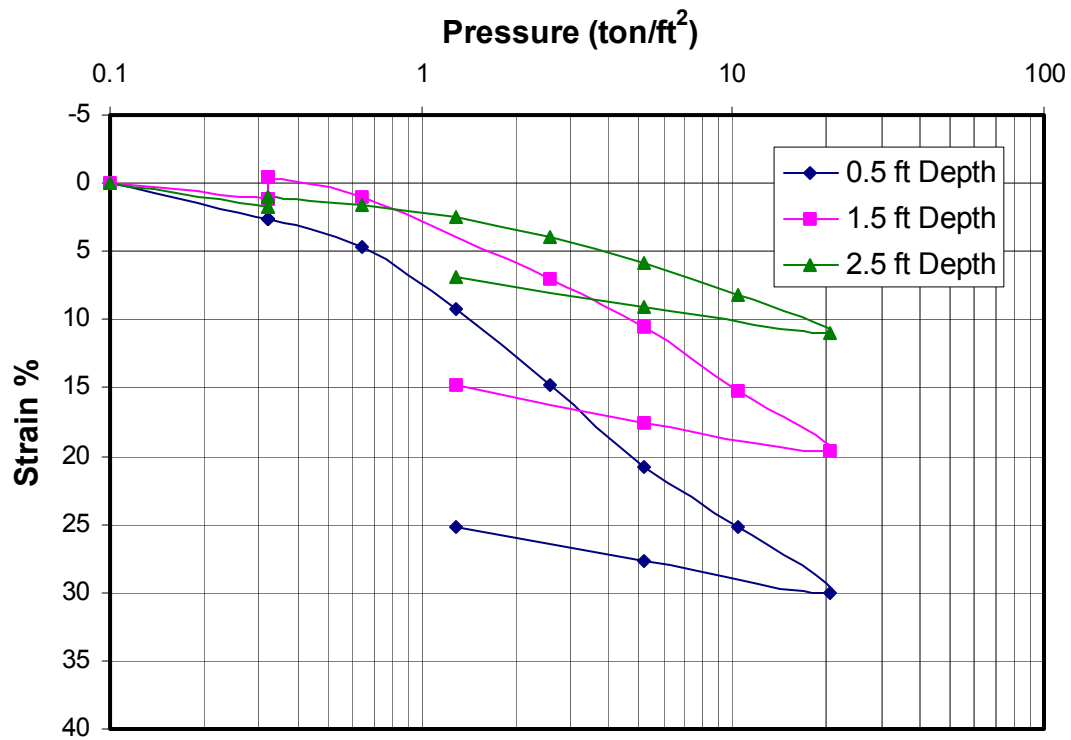


Figure 70: Strain versus applied pressure curves from consolidation tests performed on three samples from Drill Hole 6.



Figure 71: Percent swell (in negative numbers) upon wetting versus depth for samples from Drill Hole 6.



Figure 72: Shrinkage of expansive soil specimen from Drill Hole 6 (on right) after drying compared with that for collapsible soil specimen from Drill Hole 5 (on left). Both soil specimens initially had the same diameter as the steel ring.

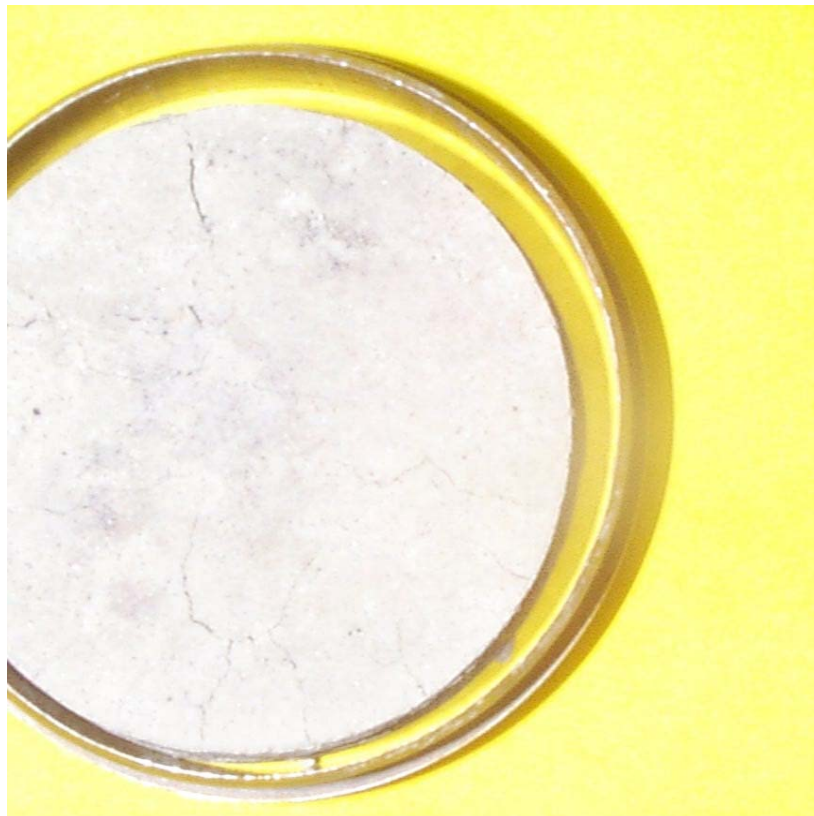


Figure 73: A close-up view of consolidation test specimen from Drill Hole 6 showing the change in diameter of the sample due to drying. The specimen fit tightly into the ring before drying.

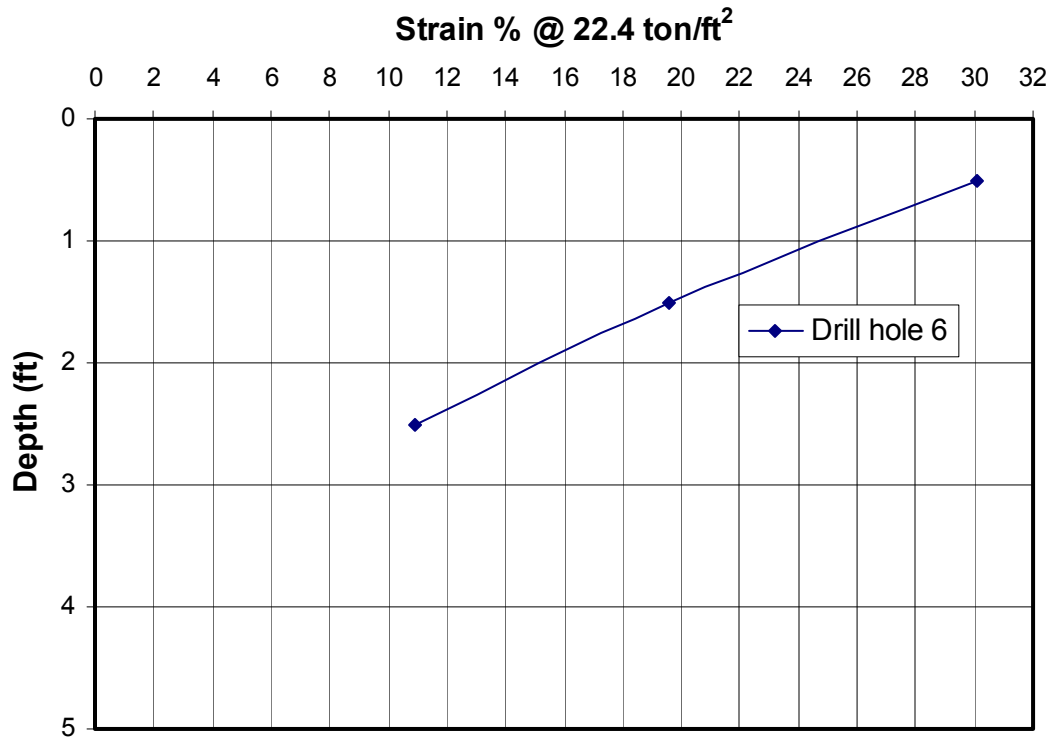


Figure 74: Percent strain versus depth at an overburden pressure of 22.4 tons/ft² for samples from Drill Hole 6.

Figure 74 shows the measured strain versus depth in Drill Hole 6 for an applied pressure of 22.4 tons/ft². Although this pressure is significantly higher than the pressure currently existing on the specimens, it does illustrate that the stiffness of the soil does increase substantially with depth. This is likely due to the fact that weathering is more pronounced at the ground surface and decreases with depth.

Figures 75 and 76 present the strain versus applied pressure curves obtained from consolidation tests on samples from Drill Holes 7 and 8, respectively. The sample at 4 foot depth in Drill Hole 7, with a PI of 21, only swelled 0.26 % upon wetting. A test could not be conducted on the sample at 1.5 feet, with a PI of 51, because the sample was very hard and extremely brittle in its dry natural state. The maximum percent swell for Drill Hole 8 was 0.98 % of the initial volume. The maximum percent collapse for Drill Hole 8 was 4.52 % of the initial volume at a depth of one foot.

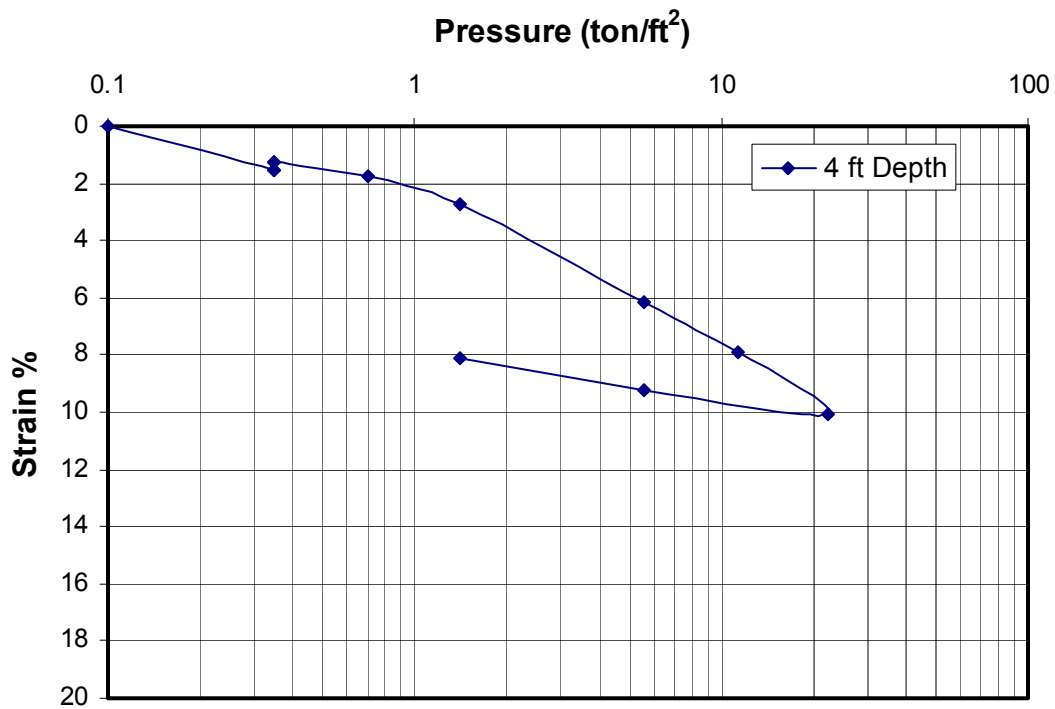


Figure 75: Strain versus applied pressure curve for test sample from Drill Hole 7.

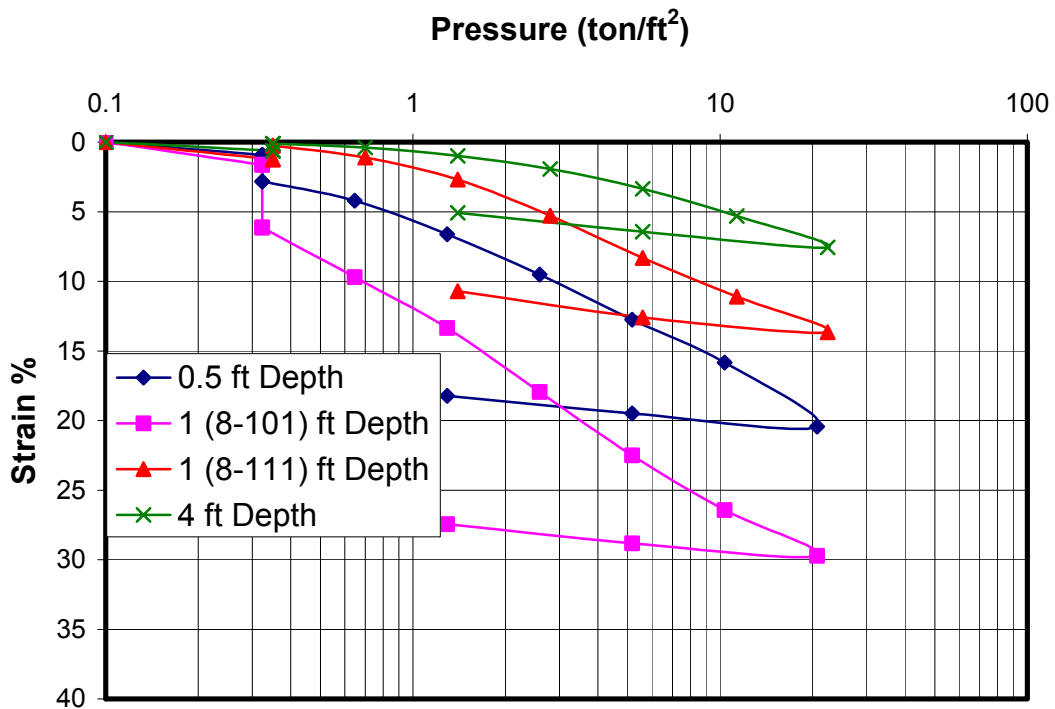


Figure 76: Strain versus applied pressure curves for test samples from Drill Hole 8.

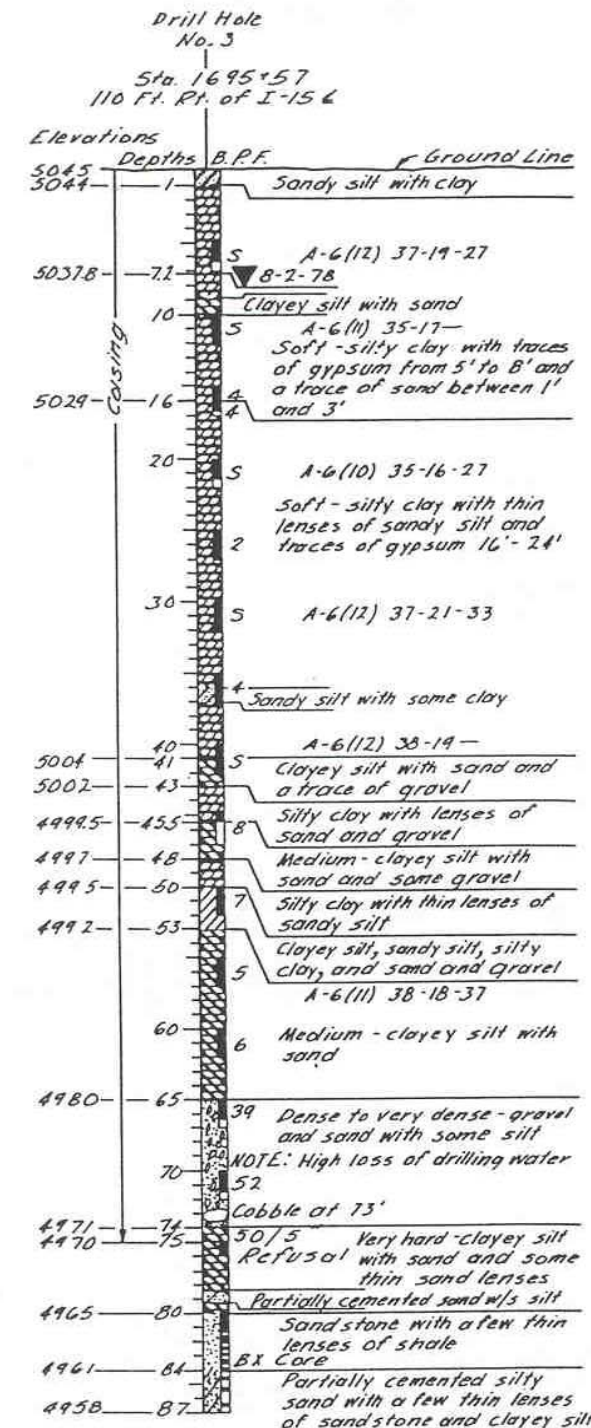
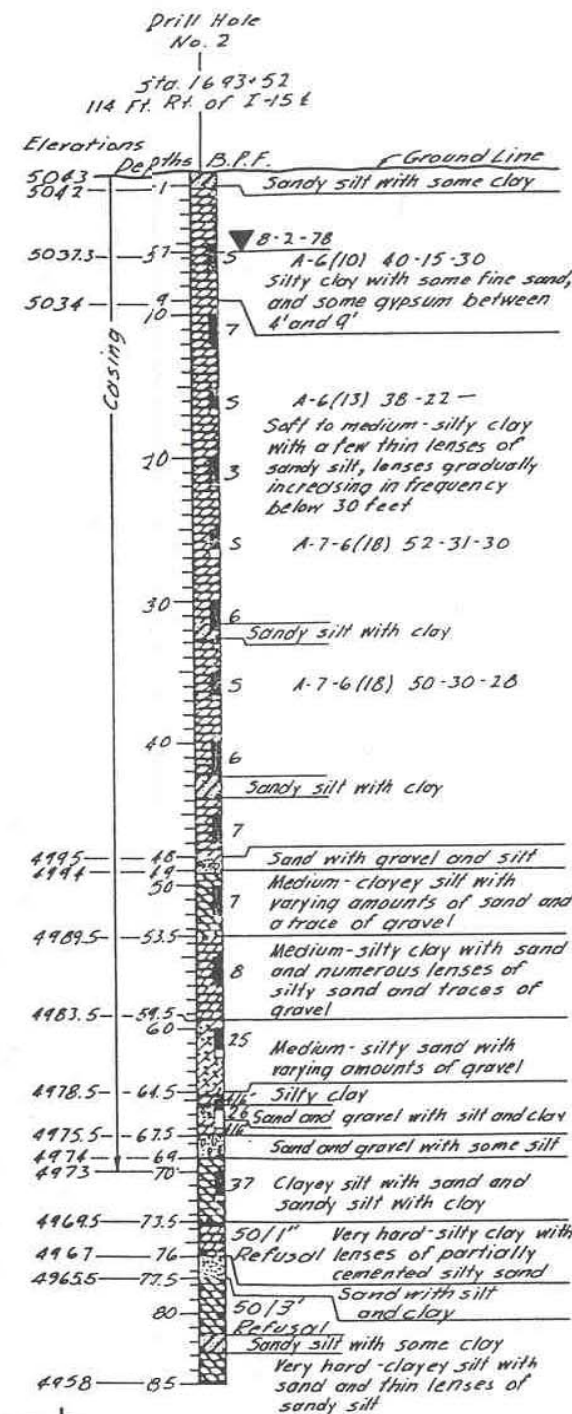
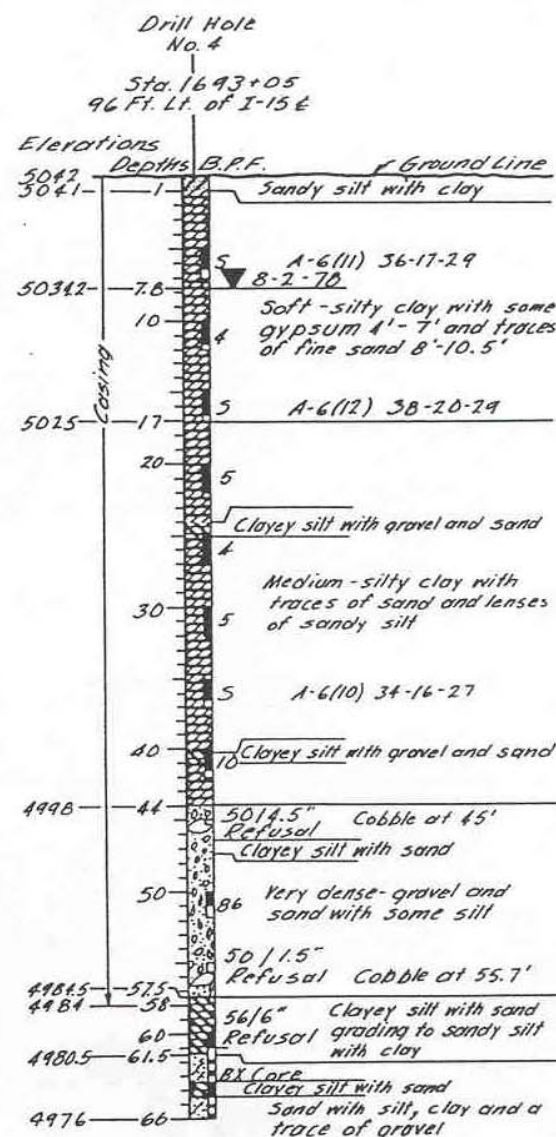
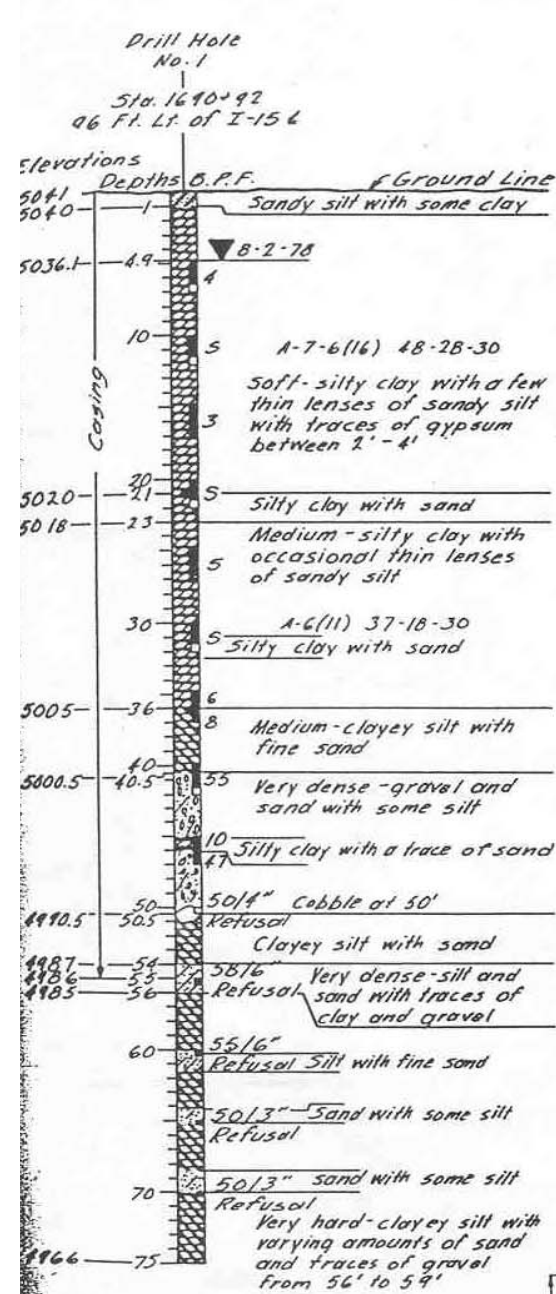
Soil Conditions Near Drill Hole 9 and UDOT Drill Holes A and M2.

UDOT Drill Holes A are located about 0.1 miles south of milepost 206. These holes are located within Qal materials on the surficial geologic map as shown in Figure 5. The boring logs for the four holes are presented in Figure 77. The soil profile typically consists of 1 foot of sandy silt which is underlain by thick layers of soft silty clay to depth of about 40 feet. The groundwater was located at 5 to 7 feet below the existing ground surface at the time of drilling. These materials typically have plasticity indices between 27 and 30. This would indicate a moderate potential for expansion. No significant distress features were observed in the immediate vicinity of these test holes.

UDOT Drill Holes M2 are located near milepost 202. These four holes are located within Qal materials based on the surficial map as shown in Figure 6. However, they are also quite close to the boundaries of a QTas zone. The boring logs for the four holes at this location are shown in Figure 78. The soil profile consists of medium to hard silty clay and clayey silts to about 80 feet below the ground. The groundwater is at 16 to 20 feet below the ground surface is quite similar to that for Drill Holes A. The plasticity index ranges from 18 to 30, which suggest that they may present minor expansion problems. In comparison with Drill Holes A the soil classification is similar; however, the water table is deeper, the soils are stiffer, and they exhibit somewhat lower plasticity.

Drill Hole 9 is located about 0.45 miles south of milepost 201 on the east side of the northbound lanes within a shallow cut section as shown in Fig. 6. The geologic units that correlate with Drill Hole 9 are QTab and QTas. Drill Hole 9 was drilled near the borderline between the two units (see Fig. 6). The boring log for the hole is presented in Figure 78. The soil profile generally consists of a layer of sandy silt about 2 feet thick which is underlain by an 8 feet thick layer of relatively plastic clay. Below 11.5 feet this clay layer is in turn underlain by layers of silty sand and gravel. The water table is located at a depth of about 15 feet. The Atterberg limits and consolidation tests on samples from Drill Hole 9 indicate that the clays may be moderately expansive. There are two edge failures next to Drill Hole 9, which could be related to an inadequate pavement section, pumping of the silts immediately below the concrete pavement or to expansion of the subsurface soils. Figures 80 and 81 are pictures of the area and distress features around Drill Hole 9. The edge of the pavement is broken into several small pieces near Drill Hole 9 and the outer northbound lane has received a surface overlay.

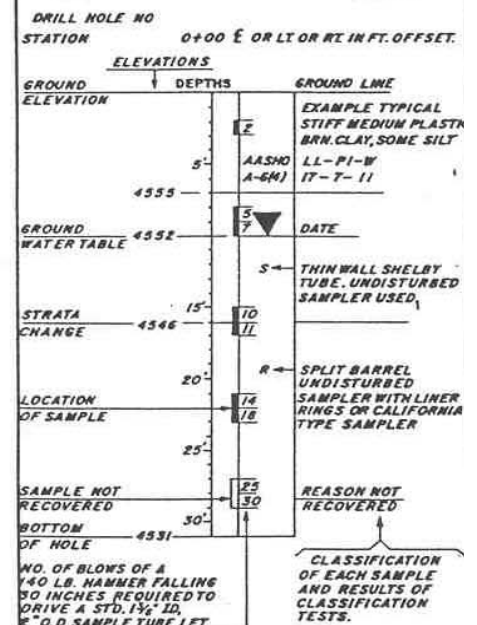
Several distress features, primarily distress at the edge of the pavement, are concentrated between UDOT Drill Holes A and UDOT Drill Holes M1 near milepost 205. These distress features are all located within a cut section about 0.6 miles long with a geologic unit defined as Tg or the Green River formation. The Green River formation consists of a limestone unit underlain by a shale unit. This shale when weathered has the potential to become expansive and may be a source of the problems at this location. Drill hole 6 is also located within this material and the soils were found to be expansive. Another drill hole in this area would be desirable.



KEY TO DRILLING LOG
RELATIVE DENSITY (NON-PLASTIC SAND & SILT)
VERY LOOSE - LESS THAN 4 BLOWS PER FOOT.
LOOSE - 4 TO 10 BLOWS PER FOOT.
MEDIUM - 10 TO 30 BLOWS PER FOOT.
DENSE - 30 TO 50 BLOWS PER FOOT.
VERY DENSE - MORE THAN 50 BLOWS PER FOOT.

CONSISTENCY (PLASTIC SILT & CLAY)
VERY SOFT - LESS THAN 2 BLOWS PER FOOT.
SOFT - 2 TO 4 BLOWS PER FOOT.
MEDIUM - 4 TO 8 BLOWS PER FOOT.
STIFF - 8 TO 15 BLOWS PER FOOT.
VERY STIFF - 15 TO 30 BLOWS PER FOOT.
HARD - MORE THAN 30 BLOWS PER FOOT.

TOPSOIL OR FILL
GRAVEL
SAND
SILT
CLAY
SHALE
IGNEOUS
LIMESTONE
CONGLOMERATE
DOLOMITE
SANDY CLAY
CLAYEY SAND
SILTY CLAY
CLAYEY SILT
SANDY SAND
SANDY SILT



ABBREVIATIONS
L.L. - LIQUID LIMIT IN %
P.I. - PLASTIC INDEX
W. - NATURAL MOISTURE CONTENT IN %
W.G. - WELL GRADED
PEN. - PENETRATION
G.W.T. - GROUND WATER TABLE
B.P.F. - BLOWS PER FOOT.
N.P. - NON PLASTIC

Note: Refusal = 50 or more blows per 6"

UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY, UTAH
MATERIALS AND RESEARCH SECTION

I-15 OVER UPRR
SOIL DATA

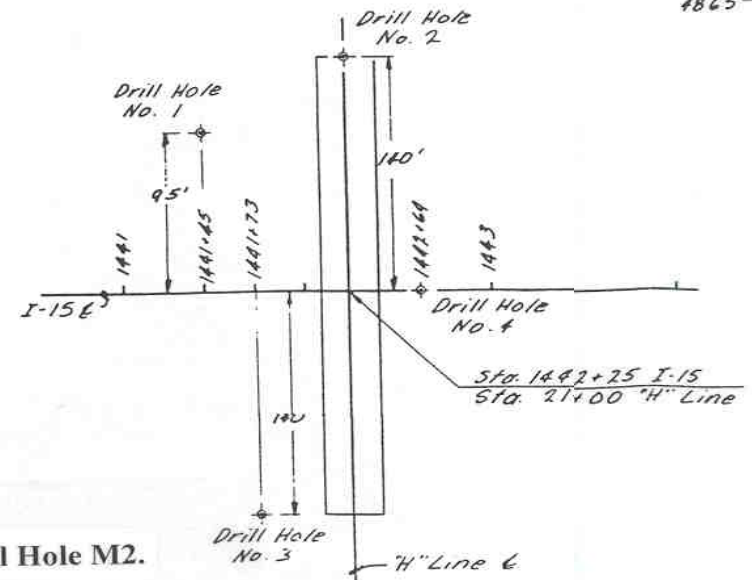
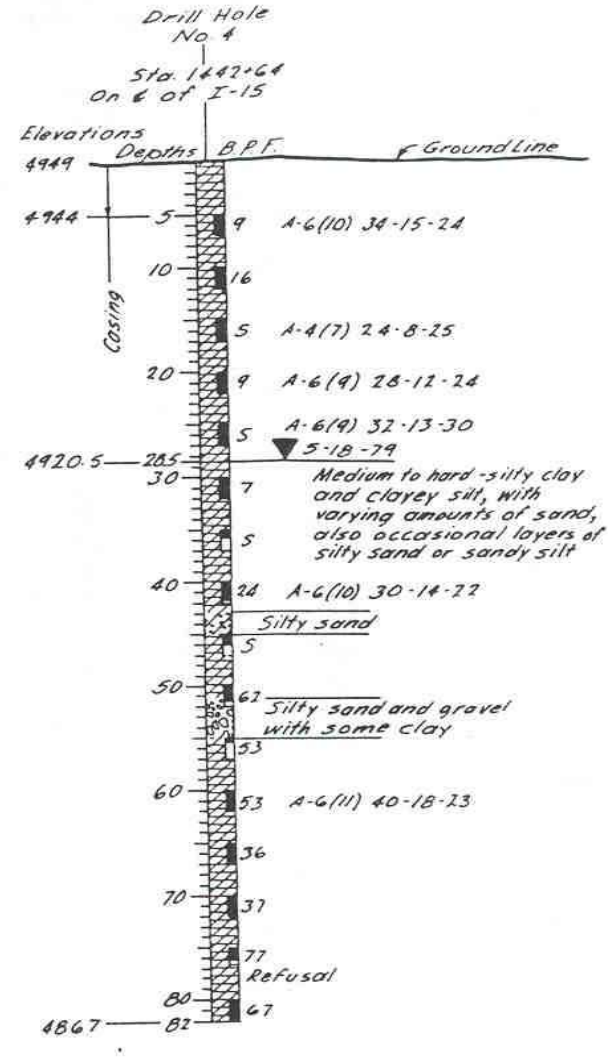
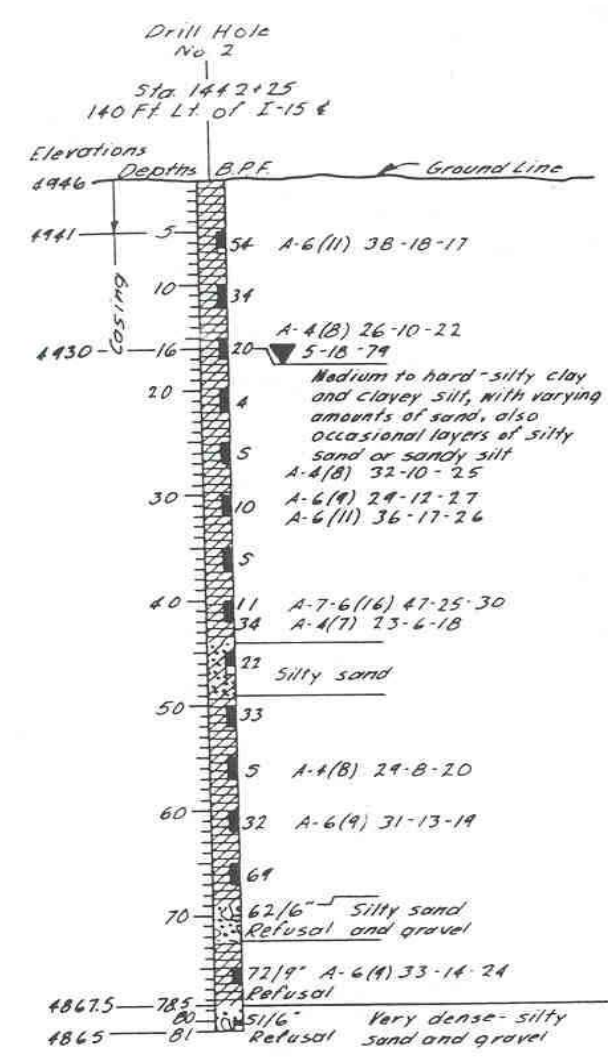
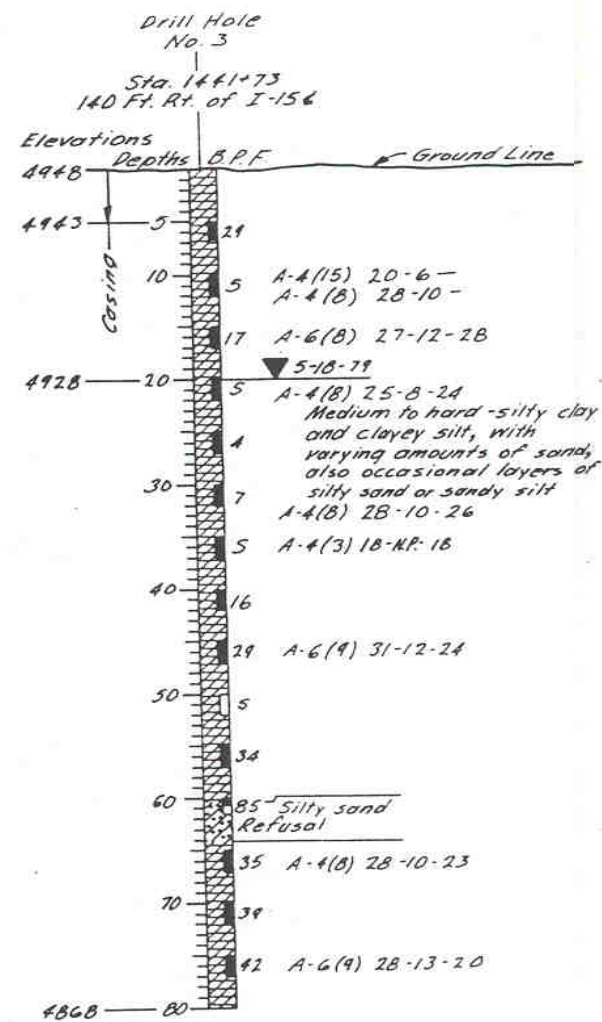
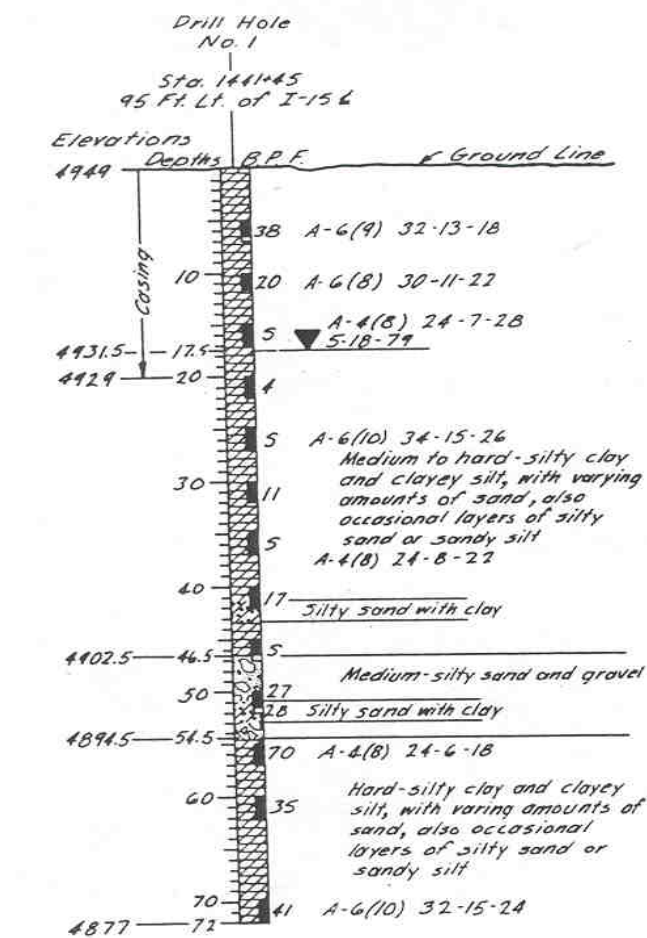
DATE DRILLED:
Drill Hole No. 1 6-7-78
" " 2 6-9-78
" " 3 7-11-78
" " 4 7-18-78

APPROVAL
RECEIVED
DATE
BY

DR. F-406 2 of 29

NOTE: Small crystals and modules of gypsum and possibly other soluble salts were encountered at the following depths:
Drill Hole No. 1: 2'-4'
" " " 4'-4'
" " " 5'-8'
" " " 4.5'-10.5'

Figure 77: Boring log for UDOT Drill Holes A.



KEY TO DRILLING

RELATIVE DENSITY (NON-PLASTIC)
VERY LOOSE - LESS THAN 4 BLS
LOOSE - 4 TO 10 BLS PER F
MEDIUM - 10 TO 30 BLS PER F
DENSE - 30 TO 50 BLS PER F
VERY DENSE - MORE THAN 50 BLS

CONSISTENCY (PLASTIC-SILT)
VERY SOFT - LESS THAN 2 BLS
SOFT - 2 TO 4 BLS PER F
MEDIUM - 4 TO 8 BLS PER F
STIFF - 8 TO 15 BLS PER F
VERY STIFF - 15 TO 30 BLS PER F
HARD - MORE THAN 30 BLS

LEGEND

TOP SOIL OR FILL
GRAVEL
SAND
SILT
CLAY
SHAPE

IGNEOUS
LIMESTONE
CONGLOMERATE
DOLOMITE

DRILL HOLE NO.
STATION
ELEVATIONS
DEPTHS

GROUND ELEVATION
GROUND WATER TABLE
STRATA CHANGE
LOCATION OF SAMPLE
SAMPLE NOT RECOVERED
BOTTOM OF HOLE

NO. OF BLOWS OF A 140 LB. HAMMER FALLING 30 INCHES REQUIRED TO DRIVE A STD. 1 3/4" ID. 2" O.D. SAMPLE TUBE 1 FT.

ABBREVIATIONS
L.L. - LIQUID LIM.
P.I. - PLASTIC IND.
N. - NATURAL MO.
W.G. - WELL GRADE
PEN. - PENETRAT.
G.W.T. - GROUND WAT.
B.P.F. - BLOWS PER F
N.P. - NON PLASTIC

Note: Refusal = 50 or more blows

e 78: Boring log and test results for UDOT Drill Hole M2.

Date Drilled: 12-78 and 5-79

NO.	BY	DATE	REVISIONS

UTAH DEPARTMENT OF TRANSPORTATION
SALT LAKE CITY
MATERIALS AND RESEARCH
SEVIER RIVER TO MILL
YUBA STATE PARK IN

DRAWN BY B. Kistler
CHECKED BY P. S. [Signature]
CHECKED BY S. SACHAI
APPROVAL
RECOMMENDED BY [Signature]
RECEIVED
DATE
LAST REVISION

BRIGHAM YOUNG UNIVERSITY 368 Clyde Building Provo, Utah 84602 Telephone: (801) 378-6334 Fax: (801) 378-4449			Boring No: BH-9 Logged By: Sizemore Date Started: 7/27/2000 Date Finished: 7/27/2000			Sheet 1 of 1 Elevation (ft): Drill Contractor: UDOT Drilling Method: Rotary w/air			NEPHI I-15												
Location			Water Level	Depth (ft)	Graphic Log	Sample Data						% Gravel	% Sand	% Fines	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits			S _u (tsf)	OTHER DATA AND REMARKS
Material Description						Location	Number	Type	Recovery (inch/inch)	Blows/6"	(N ₁₆₀) Blows/ft						Soil Classification				
																	USCS	AASHTO			
sandy silt, lt. brown, dry sandy silt with clay, brown, very damp, approaching clayey silt with sand																					
silt with a trace to some clay, grey-brown, very damp to wet Silty Clay			5			9-5	US	$\frac{24}{24}$				CH	A-7-6				26.4	69.8	24.8	44.0	
silty clay, brown, very damp to wet						9-8	US	$\frac{24}{24}$				CH	A-7-6				32.2	69.7	26.5	43.2	
			10			9-11	US	$\frac{24}{24}$				CH	A-7-6				28.1	64.4	25.0	39.4	
sand, with a trace to some silt and gravel, damp						9-14	MC	$\frac{18}{14}$	9-40-43	83						4.0					
sand with some silt and gravel to silty sand with gravel - becoming very wet - dense			15									SP	A-2-4								
			20									SP-SM	A-2-4								
silty clay, brown, saturated						9-21.5	SPT	$\frac{18}{18}$	20-22-14	36						25.3					
silty sand with some gravel, saturated												CH	A-7-6								
silty clay, with a few thin silty lenses, grey with some brown, wet - very stiff			25			9-26.5	SPT	$\frac{18}{17}$	8-12-22	34			SM	A-2-4							
												CH	A-7-6								
			30																		
MC=Modified California Sampler																					
LEGEND AND NOTES		SPT = Standard Penetration Test, Split Spoon Sample US = Undisturbed Shelby Tube, Pushed Ring = Dames & Moore Sampler Bulk = Bulk Sample From Cuttings						GROUNDWATER DEPTHS: ▽ = Initial ▴ = End of Drilling				COMMENTS: Fig. 79 Boring log and test results for Drill Hole 9.									

LOG OF BORING REV. 1 NEPHI.GPJ LANE.GDT 4/10/01



Figure 80: Looking west at the area around Drill Hole 9. Drill Hole 9 is located near the center of the picture where the person is standing. Damage along the edge of the northbound lanes of I-15 can be seen to the right of the standing person.



Figure 81: Edge damage along the northbound lane of I-15 about 0.45 miles south of milepost 201, next to Drill Hole 9.

The samples from Drill Hole 9 were taken back to the laboratory at BYU and tested. The Atterberg limits and soil classifications are shown on the boring log. The soils at depth generally have high plasticity characteristics and classify as CH or A-7-6 materials. However, there appears to be a low plasticity silt layer at the surface at least adjacent to the pavement. We are not certain that this layer exists under the entire pavement of the existing highway. Consolidation tests were also performed to evaluate the potential for expansion and unconfined compression tests were performed to evaluate the strength of the subgrade.

The measured strain as a function of applied pressure for the consolidation tests on samples from Drill Hole 9 are presented in Figure 82. The specimens were initially loaded at their natural moisture content and were subsequently saturated when the applied pressure reached the overburden pressure. Although the specimens had natural moisture contents that were relatively high, the addition of water did produce a small amount of swell. The percent swell as a function of depth below the ground surface based on the tests is shown in Figure 83. The undrained shear strength of the subgrade, measured with the unconfined compression test, is shown as a function of depth in Figure 84.

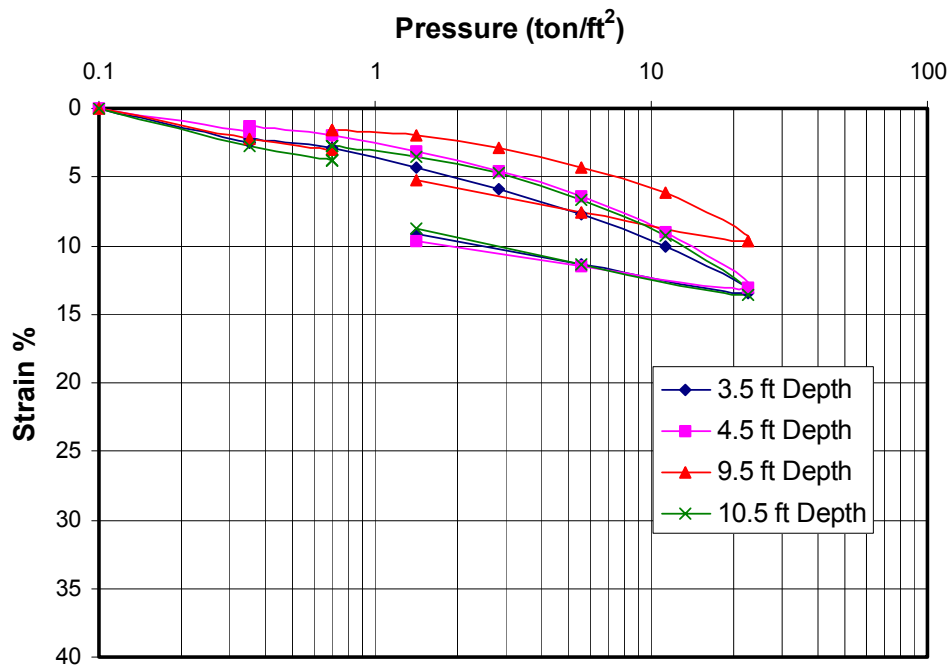


Figure 82: The percent strain versus applied pressure for samples at 3.5, 4.5, 9.5 and 10.5 foot depths in Drill Hole 9. Water was added to saturate the specimen at the overburden pressure for each sample.

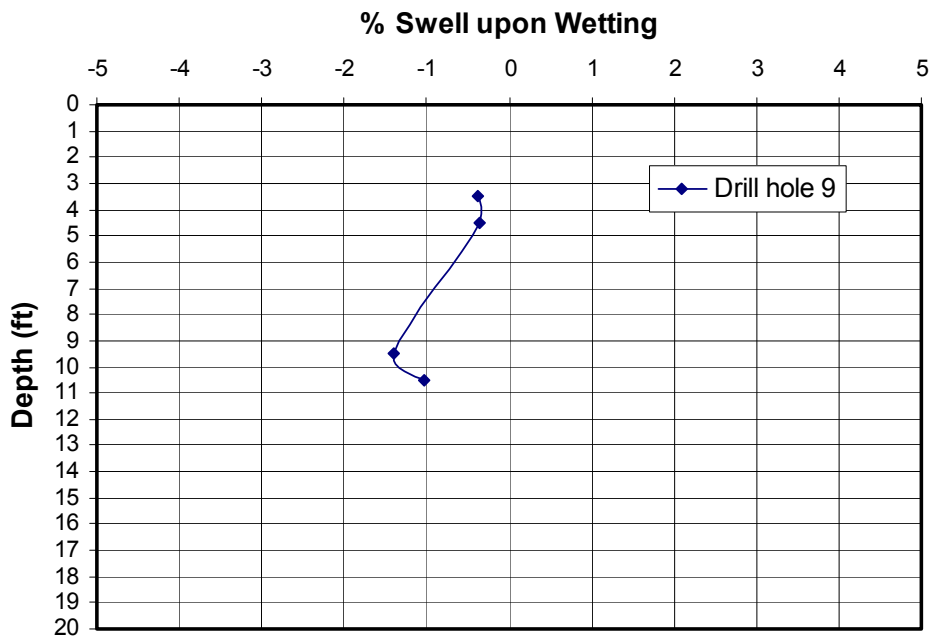


Figure 83: Percent swell (in negative numbers) as a function of depth based on consolidation tests on samples from Drill Hole 9.

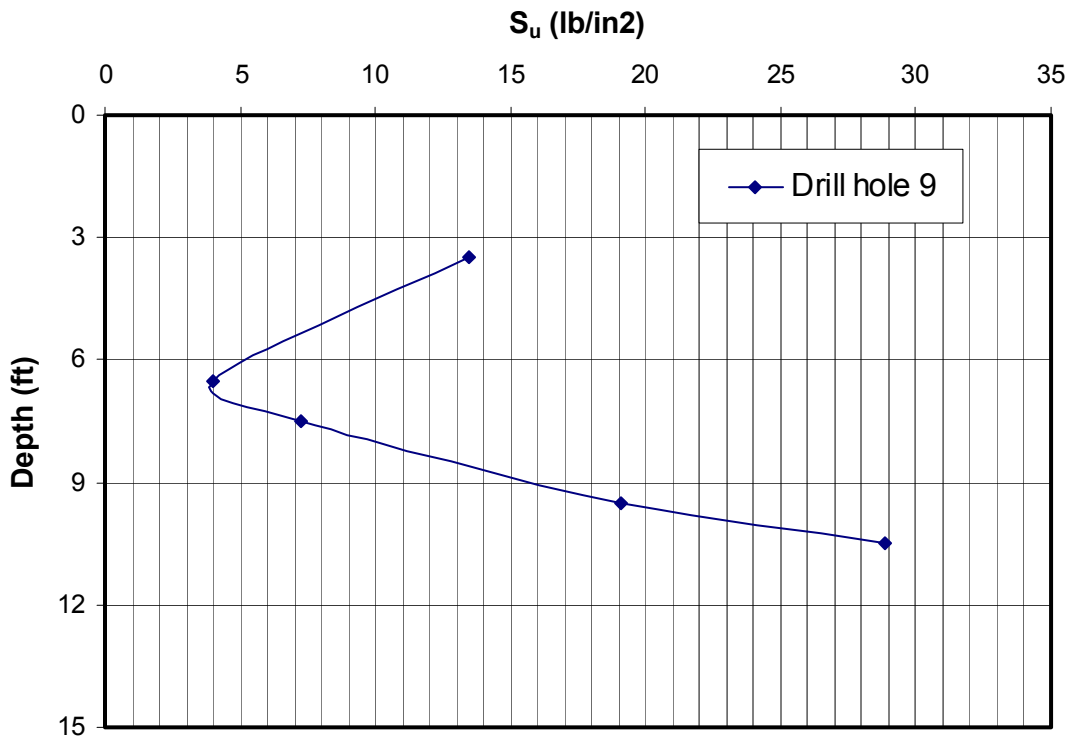


Figure 84: Undrained Shear Strength for test specimens from Drill Hole 9.

Soil Conditions Near UDOT Drill Holes M1

UDOT Drill Holes M1 are located at the Sevier River at about milepost 200. The soils are alluvial deposits designated as Qal on the geologic maps as shown in Figure 6. The boring logs for the four drill holes are presented in Figure 85. The soil profile generally consists of a surface layer of sandy silt with some clay which is 3 to 5 feet thick. This layer is underlain by a medium to dense silty sand layer about 28 to 30 feet thick. This silty sand layer is in turn underlain by a dense silty sandy gravel which extended to the bottom of the drill holes. The water table was typically 5 to 7 feet below the existing ground surface. No significant pavement distress is evident in the vicinity of these drill holes.

CBR Testing

To facilitate the pavement design process, standard Proctor tests (AASHTO T-99) and nine California Bearing Ratio (CBR) tests were performed on nine samples taken from the subgrade materials within the study area. A summary of the test results is provided in Table 2. The materials were typically fine-grained soils ranging from silts to plastic clays. The percent swell upon wetting under the load from a 10 lb weight is also shown for each sample. The swell was typically 0.7 to 0.9 % for the silts, but ranged from 1.1 to 4.7 % for the clays. CBR values ranged from 1.0 to 8.0 and there was no apparent correlation with soil type.

Table 2: Summary of Standard Proctor tests and CBR tests for subgrade samples at various locations along the study area.

Milepost Location	Material Type	Swell On Wetting (%)	CBR Value (%)	Optimum Moisture (%)	Max. Proctor Density (lb/ft³)
200.523	Light Brown Silt	0.8	8.0	15.3	111.3
203.806	Light Brown Silt	0.7	4.5	14.0	112.2
207.292	Brown Clayey Silt	0.9	3.7	15.5	111.9
207.765	Greenish Gray Clay	1.1	3.3	19.2	108.4
208.820	Yellowish Brown Clay	4.7	1.0	26.5	91.7
209.007	Light Brown Silt	0.87	3.5	18.6	104.4
209.280	Greenish Gray Clay	1.25	4.0	27.0	91.7
209.833	Reddish Brown Silt	0.8	2.8	18.7	106.3
210.622	Silty Clay	1.7	5.7	23.4	97.3

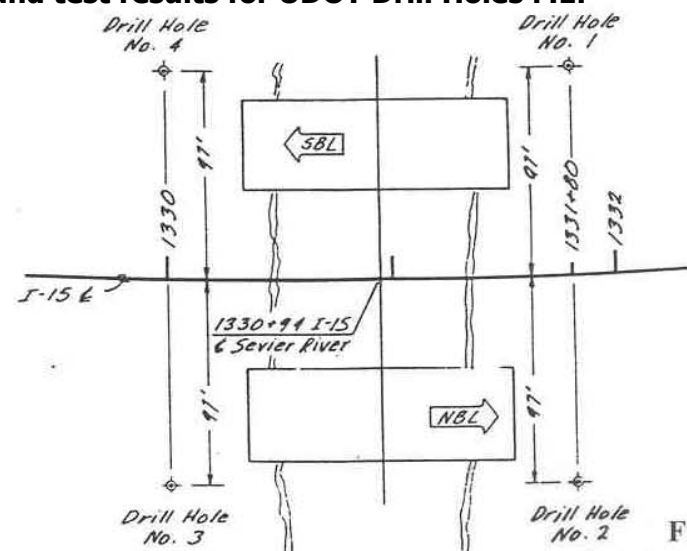
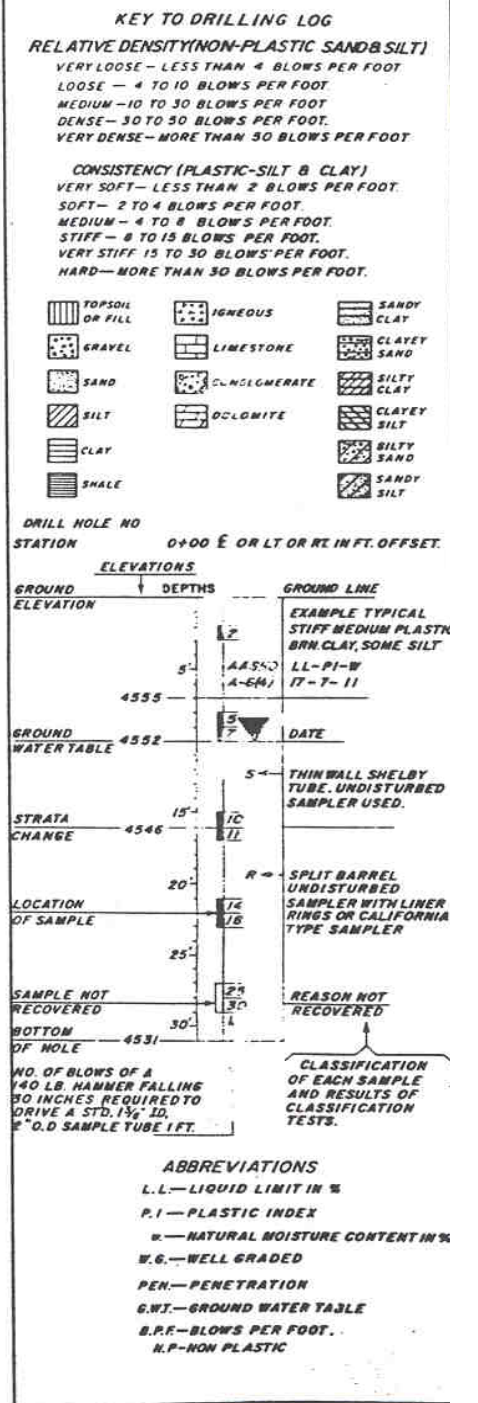
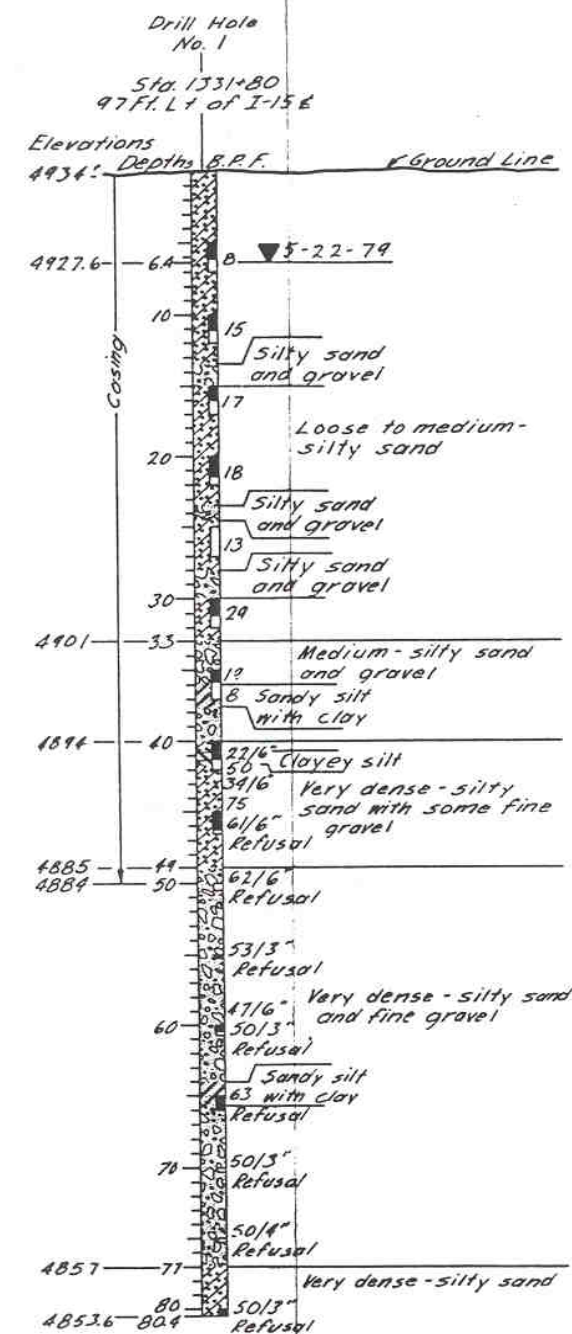
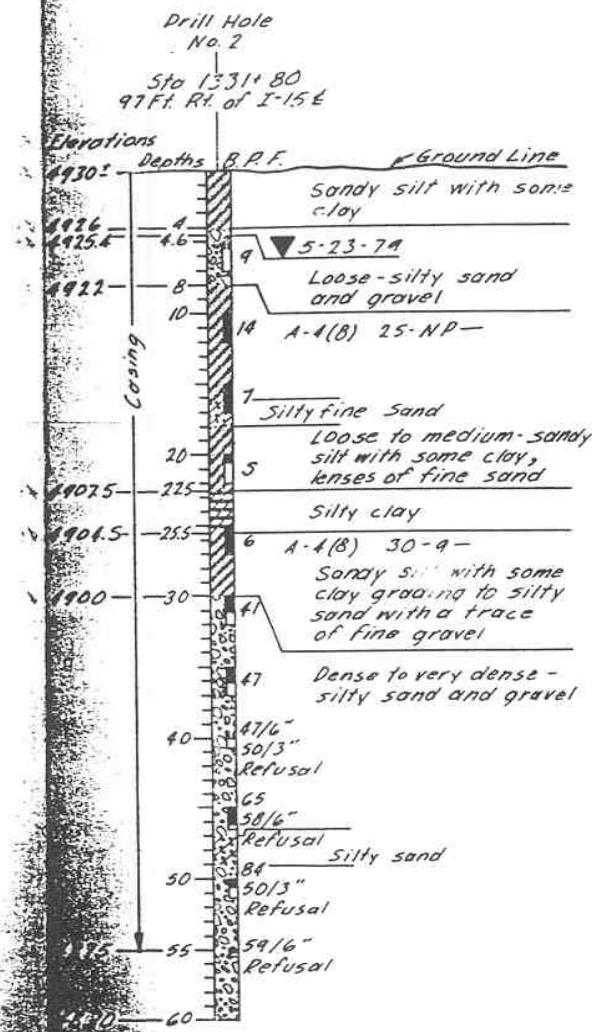
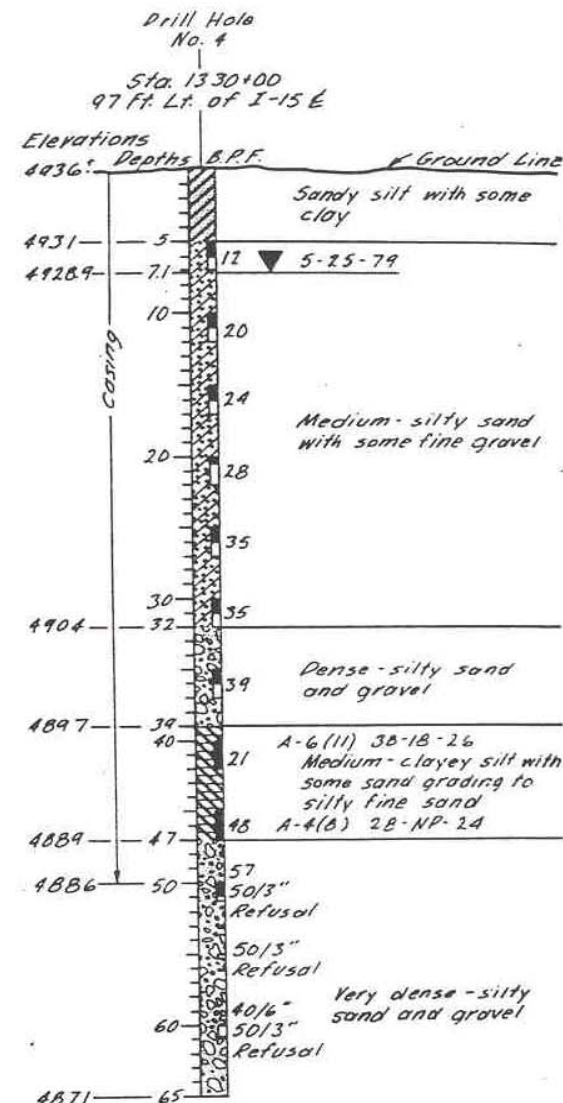


Figure 85: Boring logs and test results for UDOT Drill Holes M1.

4. DESIGN APPROACHES FOR MITIGATING THE DISTRESS

Collapsible Soil Treatment Areas

Based on the distress mapping, geologic mapping and soil exploration results, collapsible soils appear to be a significant cause of concentrated distress in zones lying in alluvial fan materials near boreholes 2 and 5. Test borings and laboratory testing indicate that the collapsible soils in these areas extend to between 15 to 20 feet below the ground surface. The boundaries for these four zones are shown with dashed black line in Figures 86 and 87. Figure 86 provides an overall view of the zones and the clay-bearing rock in the upstream drainage basins upstream from the alluvial fans which are conducive to collapsible soil formation. Figure 87 provides a close-up view which shows the zones relative to the borehole locations and distress points. The combined length of the four zones is 0.735 miles. The first zone starts, for both south and northbound lanes, at mile 209.728 and ends at mile 209.412 for a length of 1670 feet. The second zone, starts at mile 209.308 and ends at mile 209.245 for a length of 332 feet. The third zone starts at mile 209.22 and ends at mile 209.153 for a length of 352 feet. The fourth zone, for both the south and northbound lanes, starts at mile 209.108 and ends at mile 208.814 for a length of 1550 feet. These boundaries will likely need to be refined during fieldwork as more information on the exact boundaries is uncovered. The present boundaries are somewhat conservative and extend through the fan material and into the cut sections in some cases.

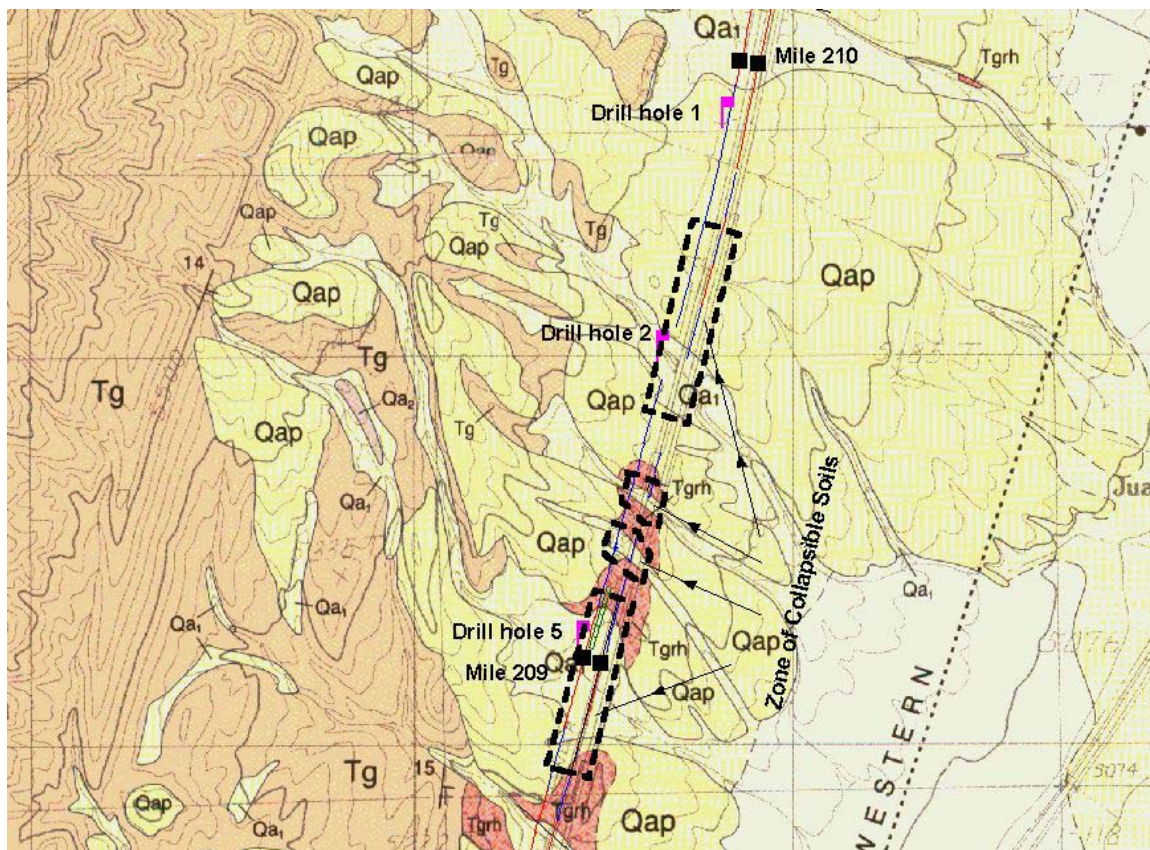


Figure 86: Map showing the four zones where collapsible soils have led to concentrated damage in the vicinity of milepost 209 and test holes 2 and 5. Lithology in the drainage basins upstream from the alluvial fans is conducive to the formation of collapsible soil.

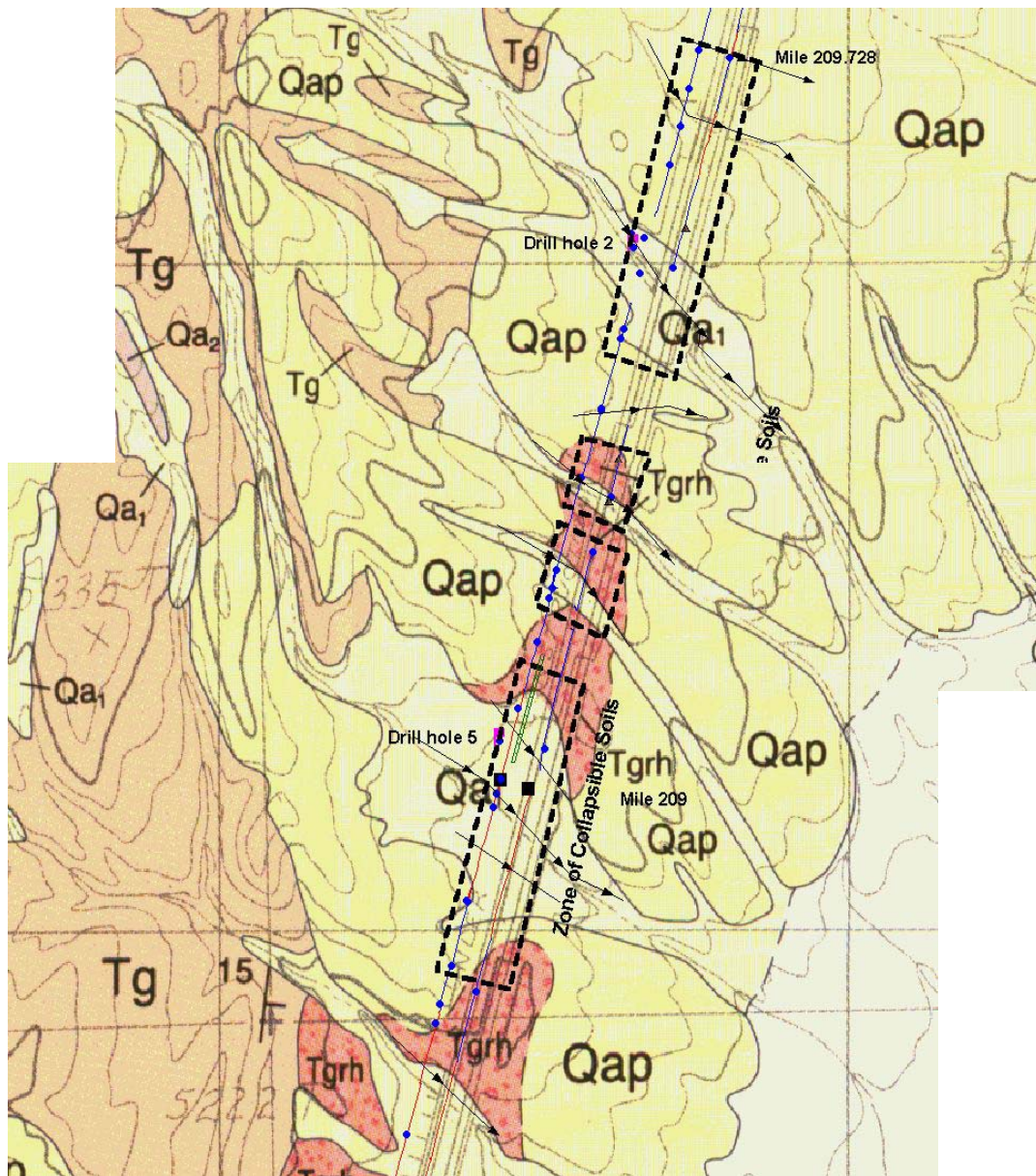


Figure 87: Map showing Drill Hole 2 and the distress features likely related to collapsible soils. The dashed black boxes represent the proposed length of highway for treatment.

Collapsible soils are likely present in the alluvial fans north of the proposed treatment zones, however, the relatively sporadic distress observed over this length of the study area over the 10 to 15 year service life does not appear to justify remedial treatment measures.

Potential Treatment Approaches for Collapsible Soils

There are several treatment options for dealing with the collapsible soils hazard identified in this study. These options include: (1) Continuously deal with collapse settlement as it occurs using maintenance funds, (2) Surface drainage to prevent wetting, (3) Pre-wetting the soil prior to reconstruction, (4) Excavation and recompaction, (5) Deep dynamic compaction. Each of these options will be discussed separately and then the recommended approach will be discussed in more detail.

(1) Continuously deal with collapse settlement as it occurs using maintenance funds.

This is the approach that has been utilized over the past 10 to 15 years since the road was constructed. Maintenance funds have generally been insufficient to keep the roadway leveled and up to expected service levels for an interstate highway. Laboratory tests indicate that significant collapse settlement (1 to 5 % by volume) can still occur in the subgrade soils when they become wet in the future. This suggests that the maintenance problem will remain a concern for some time to come. If this option is pursued, additional maintenance funds and personnel should be allocated to deal with future problems.

(2) Surface drainage to prevent wetting.

This option would be desirable in combination with any of the other options that might be selected. While this approach should theoretically eliminate the collapse, this option is generally more difficult to implement from a practical standpoint than might generally be supposed. Experience has shown that extreme events or failure of drainage systems generally produce wetting events which lead to concentrated collapse settlement somewhere along the roadway. This concentrated settlement leads to differential movement and cracks or bumps in the roadway. Therefore, this option, while beneficial, cannot generally be relied upon as the single solution to the hazard.

(3) Excavation and re-compaction.

The collapsible soils at this site generally consist of silty sands and sandy silts. These materials can usually be excavated and then re-compacted to provide a relatively dense subgrade for the highway. This option is generally more economical when the depth that must be replaced is on the order of 5 feet thick or less, which is not the case at this site. If partial excavation and replacement is used, then a calculated risk is assumed. The compacted fill zone will increase the thickness of soil which surface water must penetrate in order to produce settlement in the collapsible soil at depth. The compacted soil layer also tends to reduce differential settlement at the surface. Finally, there is less collapsible soil remaining to cause settlement and collapse strains usually decrease with depth. However, if water does penetrate into the remaining collapsible soil, settlement can still occur and lead to pavement distress.

(4) Pre-wetting prior to reconstruction

This option has been tried at a number of sites where no additional load was required in order to induce collapse settlement prior to construction of a roadway or canal. However, results in practice have been mixed. Surface infiltration does not always guarantee that water will penetrate uniformly to the full depth of collapsible soils, particularly if a relatively impermeable layer is encountered. These problems can be overcome to some extent by drilling vertical holes in a grid pattern, but this substantially increases the cost of the operation. Creep settlement after wetting has been observed in previous case histories and this could lead to significant post-construction settlement.

(5) Deep Dynamic compaction

Deep Dynamic compaction (DDC) is the process of densifying soil by repeatedly dropping a large weight with a crane. The weight is dropped in a grid pattern at the surface and improvement can be produced to depth of 30 feet. Drop heights typically range from 50 to 80 feet and drop weights are generally between 10 to 25 tons. DDC has been employed at 12 collapsible soil sites in the western United States as shown in Figure 88 since 1984 with reasonably good success. Seven of these projects have involved remedial treatment work for interstate highways in New Mexico, Wyoming and Montana. Soil conditions were similar to those at this site (Rollins et al, 1999). DDC is the most inexpensive soil improvement method presently available.

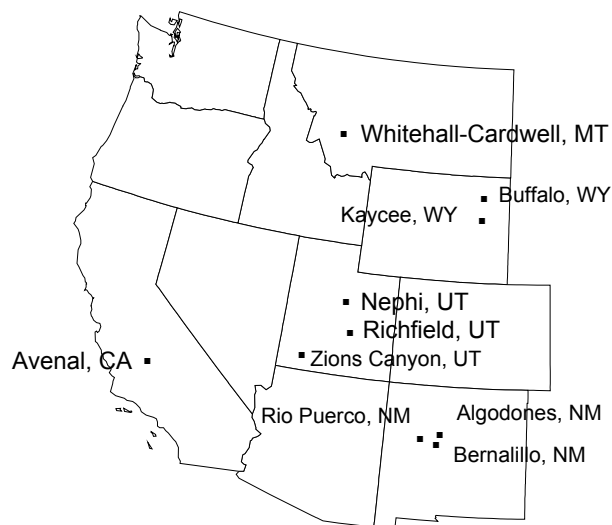


Figure 88: Location of deep dynamic compaction projects involving collapsible soils in the United States.

Recommended Treatment Approach for Collapsible Soils Zones

Based on the thickness of the collapsible soil identified during this study, we recommend that deep dynamic compaction be used to treat the collapsible soils in the four zones identified previously. In these zones, the current distress has been significant enough that we consider positive soil densification to be appropriate. As indicated previously, laboratory collapse tests demonstrate that despite previous settlement the existing soil profile is still capable of inducing significant collapse settlement from future wetting episodes. DDC is relatively inexpensive considering the improvement that is produced in-situ without the need for significant earth moving. In addition, the performance of highways on collapsible soils treated in this manner has been relatively good (Rollins and Kim, 1994; Rollins et al., 1999).

Additional Background on Deep Dynamic Compaction

Depth of Improvement

The depth of improvement is the depth below which there is no improvement in penetration resistance or soil density following treatment. Below this depth, the density after compaction is the same as that before compaction. The depth of improvement, D in meters is typically computed using the equation

$$D = n (WH)^{0.5} \quad (1)$$

where W is weight in metric tonnes and H is drop height in meters (Lukas, 1986). Lukas (1986) suggested that 0.5 was a reasonable first approximation for the n value and listed recommended n values for different soil types. Based on a study of 10 DDC case histories in collapsible soils, Rollins et al (1999) recommended an n value of 0.40 although significant variation in the depth of improvement was observed. Back-calculated n values ranged from 0.2 to 0.6. Because of this uncertainty, a pilot test program is normally performed at the project site to confirm performance before the final production program is undertaken.

Rollins et al (1999) found that most of the significant improvement occurred in the upper two-thirds of the depth of improvement. Based on this assessment, the D used in Eq. 1 should be selected so that significant improvement occurs in the zone where collapse potential must be reduced. We would, therefore, recommend that D be taken as 8 meters or 25 ft for preliminary estimation purposes. This could likely be achieved using a 16 tonne weight dropped from a height of 25 meters or 82 feet.

Drop Spacing

Dynamic compaction is typically performed by dropping a tamper on a rectangular grid pattern. At each drop location, 5 to 15 drops may be made. Large drop spacings are typically employed initially. After treating the entire area once, successive passes may be carried out at intermediate points to treat the area more evenly. Chow et al (1994) showed both experimentally and theoretically that the degree of improvement and uniformity of improvement is a function of drop spacing. They showed that if the final center-to-center spacing is about two times the diameter of the tamping weight then the improvement at the center of the grid is

about the same as that below the drop point. Spacing to diameter ratios of two have commonly been employed in treating collapsible soils (Rollins and Kim, 1994).

Tamper Shape and Contact Pressure

The surface area of the tamper that impacts the ground may be round, square, hexagonal or octahedral. Generally, rounder shapes are more efficient since the tamper can drop into the crater formed by previous drops without impacting the sidewall and losing energy. The contact pressure is the weight of the tamper divided by the surface area that impacts the ground. Previous DDC treatment of collapsible soil has typically employed contact pressures between 40 and 80 kPa. If the pressure is too high, the tamper punches through the ground without densifying the surrounding soil, but if it is too low the surface layers are densified without much improvement in the deeper soil layers.

Energy Requirements

Energy for compaction is usually expressed as unit energy applied over an area of the ground surface or as the applied energy divided by the volume of soil tested. Lower energy values are typically appropriate for soils which are initially denser and higher values are usually required for the looser soil profiles. Rollins and Kim (1994) found that the energy typically used in densifying collapsible soils was between 37 and 70 t-m/m³ for nine projects in the western U.S. For comparison purposes, the Standard Proctor energy is 60.5 t-m/m³. Compaction energy for dynamic compaction of collapsible soils is typically greater than for dynamic compaction of non-collapsible soils particularly if the soil is dry (water content less than 10%).

Number of drops

Based on the drop height and the energy requirements, the number of drops can be estimated. Typically, the number of drops at one location is less than 10. The number of drops can be modified during construction by monitoring the depth of crater formed by the tamper and the heave of the surrounding soil. When the heave volume exceeds the volume of the crater, the dynamic compaction is no longer causing any net improvement and the compaction can be terminated. Several compaction projects have also terminated compaction when the increase in crater depth was less than 10% of the total crater depth.

Pilot Testing program

Because of the uncertainties involved in dynamic compaction work, a preliminary testing program is normally carried out to evaluate the design assumptions. The specialty contractor will normally propose a treatment program and perform the work. The improvement in either penetration resistance or collapse strain will then be evaluated. If the method is unsuccessful, additional energy will be applied and the soil conditions re-tested. This procedure is then repeated until a successful approach is developed. When this approach is approved, the rest of the compaction work will typically be carried out using the successful treatment method. Inspection work in this case will generally consist of ensuring that the required energy is applied. When the soil conditions along a site vary from location to location, a second test area may be used or test borings/samples may be taken at some interval along the length of the treatment area to ensure that the method is producing the desired results within the range of

materials encountered. At this site, test sections may be desirable in the vicinity of the test borings drilled for this study where soil profile and collapse strain information could be used to provide a reference to the test results following treatment.

Cost Considerations

Typical costs for DDC include the cost for a test section, inspection, mobilization and production tamping. Typical DDC production costs have been in the range of \$1.00 to \$1.25 per square foot of surface area with a mobilization cost of approximately \$30,000. Most specialty contractors prefer to perform the test section work immediately preceding the production work in an effort to minimize mobilization costs.

Expansive Soil Treatment Areas

Distress related to expansive soils exists throughout the study area, but significant damage concentrations are located in a cut section between mileposts 208 and 207 along I-15. This area is long enough to propose treatments for the area, in order to improve ride quality throughout the cut section. Other areas are isolated and are generally too minor to be considered for treatment proposals. The map in Figure 89 shows the expansive soil zone which is recommended for treatment along I-15 and also includes the distress features and drainage paths throughout the cut section. The length of the treatment zone is 3209 feet in the Southbound lane (0.608 miles, Milepost 207.939 - 207.331) and 1779 ft in the Northbound lane (0.337 miles, Milepost 207.939 - 207.602).



Figure 89: Expansive soil zone proposed for treatment and the distress features throughout the cut section between mileposts 208 and 207 along I-15, Juab County, Utah.

Potential Treatment Options for Expansive Soils

A number of treatment options have been investigated and employed for highways on expansive soils in the United States. Snethen (1979) and Nelson and Miller (1992) provide useful summaries of the performance of these methods and make recommendations for future projects involving roadways on expansive soils. Methods for dealing with expansive soils

include: (1) Continuously deal with volume change as it occurs using maintenance funds, (2) Control subgrade moisture conditions, (3) Subexcavation and replacement, and (5) Chemical alteration. Each of these options will be discussed separately and then the recommended approaches will be discussed in more detail.

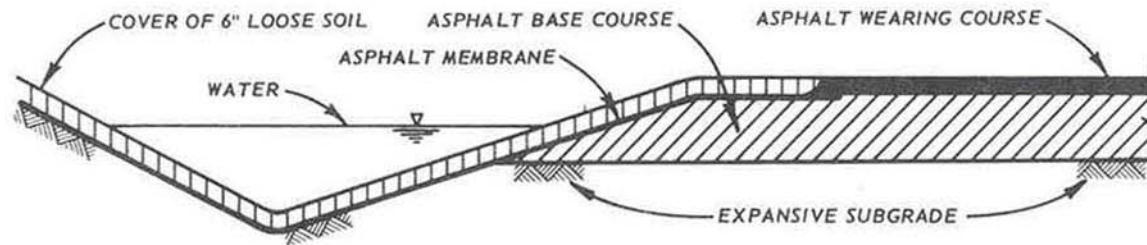
(1) Continuously deal with volume change as it occurs using maintenance funds.

This is the approach that has been utilized over the past 10 to 15 years since the road was constructed. Maintenance funds have generally been insufficient to keep the roadway leveled and up to expected service levels for an interstate highway. Laboratory tests indicate that the expansive soils in the subgrade will continue to experience significant volume changes of (+5% (shrinkage) to -2% (swell)) as the moisture content fluctuates with the seasons. When the soils become wet in the winter and spring, expansion will occur and when the soils dry out in the summer, the soils will shrink. These expansion and contraction cycles will continue to stress the pavement and produce a continual maintenance problem. If this option is pursued, additional maintenance funds and personnel should be allocated to deal with future problems.

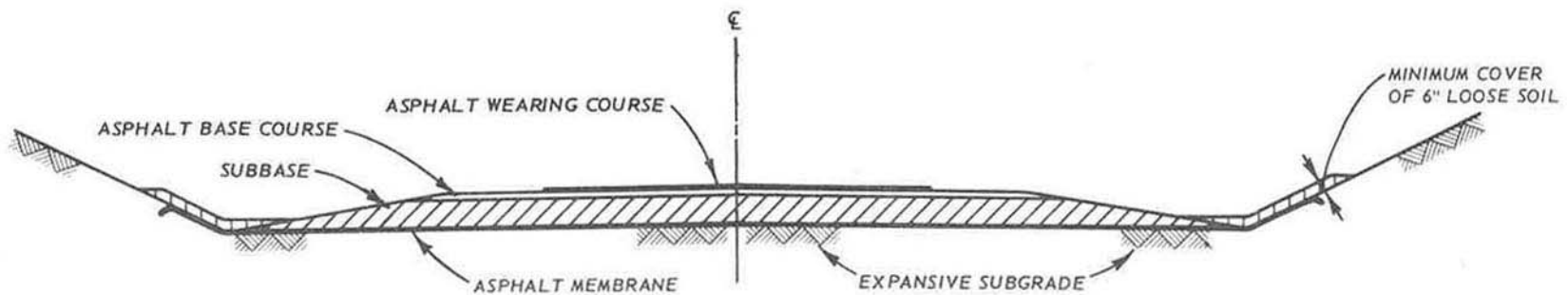
(2) Control subgrade moisture conditions.

Since volume change of expansive soils results from changes in subgrade moisture content, controlling the variation in moisture content can help control the volume change. Waterproofing membranes have been successfully used to control moisture content for a number of highways in Colorado, Texas and Arizona (Snethen, 1979). Membranes have consisted of (a) continuous sprayed asphalt membranes over the entire subgrade and ditches, (b) full-depth asphalt pavement with a sprayed asphalt or synthetic fabric membrane beneath the ditch, (c) full-depth asphalt pavement with paved ditches in cut sections, and (d) vertical synthetic fabric membrane cutoffs. Waterproofing membranes have typically consisted of continuous sprayed asphalt (catalytically blown, emulsified, or asphalt rubber). Snethen (1979) notes that membranes "perform best in situations where the soil profile is relatively dry: the moisture content profile is relatively uniform with depth; the groundwater table is at a sufficient depth and has no influence on near surface behavior; and the climate is dry-subhumid or drier". All of these conditions appear to be met within the study area suggesting that the use of membranes would be effective. Figure 90 shows typical applications of continuous sprayed asphalt membrane and full-depth asphalt pavement with sprayed asphalt membrane beneath the ditches. The sprayed asphalt layers outside the paved area all receive a 6" cover layer of soil to protect them from damage. Snethen (1979) recommends that the membrane cover the median in divided four-lane highways if physically and economically feasible.

Vertical edge membranes have also been used along with horizontal membranes when significant lateral inflow of water is anticipated as shown in Figure 90. However, since many expansive soils have relatively low permeability, this is not always a problem. Experience indicates that vertical drains by themselves are not effective in maintaining a constant water content long-term. Vertical membranes less than 3 feet deep have not proven effective and depths equal to the active zone (zone exhibiting moisture fluctuation) are recommended. Since this may require depths of 10 to 15 feet in some cases, economics or practical construction limitations may limit membrane depths to one-half or two-thirds of the active zone. After placing a geosynthetic membrane in the trench, the trench should be backfilled with low permeability materials as shown in Figure 91.

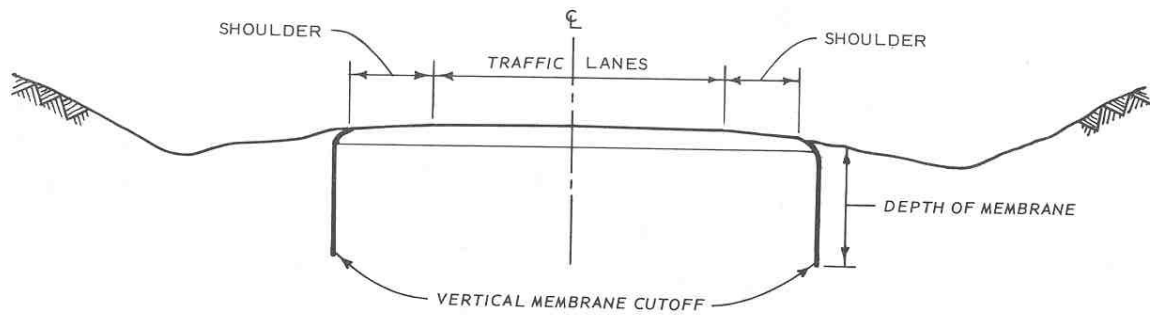


FULL-DEPTH ASPHALT PAVEMENT WITH LINED DITCHES



CONTINUOUS ASPHALT MEMBRANE APPLIED TO SUBGRADE AND DITCHES

Figure 90: Schematic drawing of two method for constructing surface moisture barrier to prevent infiltration of water into the expansive soil subgrade (Snethen, 1979).



NOTE: THE DEPTH OF THE MEMBRANE CUTOFF SHOULD EXTEND TO THE DEPTH OF ACTIVE ZONE; HOWEVER, IF THE SOIL CONDITIONS AND THE PRACTICALITY AND ECONOMICS OF THE INSTALLATION ARE NOT FAVORABLE THEN THE DEPTH SHOULD BE REDUCED TO FACILITATE INSTALLATION.

Figure 91: Schematic drawing showing vertical membrane cutoff on either side of highway to prevent lateral migration of water into expansive soil subgrade (Nelson and Miller, 1992).

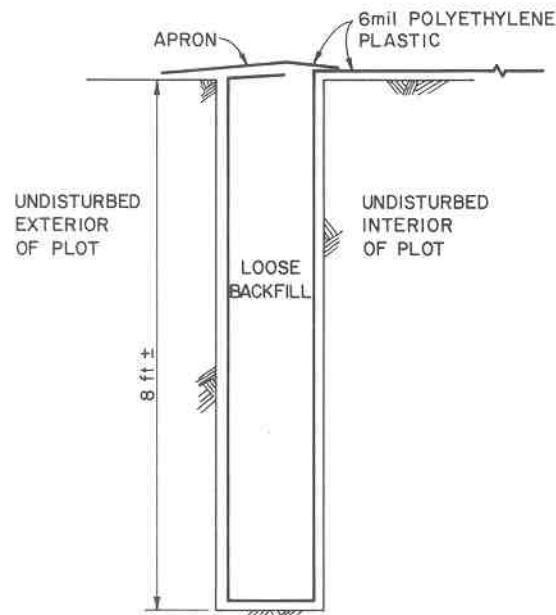


Figure 92: Schematic drawing showing detail of vertical membrane cutoff with liner material backfilled with cohesive soil. (Goode, 1982).

Edge drains consisting of gravel or synthetic drain materials have not proven effective in dealing with expansive soils. These edge drains typically produce more harm than good and eventually introduce water into zones where it would not otherwise penetrate.

(3) Subexcavation and replacement.

Subexcavation and replacement involves the removal and replacement of the expansive subgrade soil with non-expansive material. The compacted backfill selected for the replacement should not cause problems with respect to the in-situ soil. According to Sneden (1979), granular soils should never be used as backfill for subexcavation and replacement projects. Granular soils provide access for surface water to penetrate into the subgrade and tend to function more as a reservoir than as a drain. Therefore, fine-grained backfills with low-permeability should be selected. The soil being removed can be used as a backfill provided that it is chemically or mechanically altered to reduce the swell potential. This is typically accomplished using lime treatment as discussed subsequently. The replacement option offers three benefits. First, the surface layer of the subgrade which normally experiences the greatest fluctuation in water content will no longer be expansive. Second, although expansive material is still in the profile, it will be located at greater depth and below a relatively impervious layer where water content fluctuation is less likely. Finally, the compacted backfill will serve as a surcharge on the underlying native soil and will tend to minimize differential movement.

The benefits of the subexcavation method increase as the thickness of the compacted fill layer increases. Chen (1988) recommends a minimum thickness of 3 to 4 feet, while the Colorado DOT recommends a thickness based on the Plasticity Index of the subgrade as shown in Table 3 for interstate highways (Safford and Egger, 1974).

Table 3: Recommended depth of subexcavation and replacement layer based on subgrade plasticity index (Safford and Egger, 1974)

Plasticity Index of Subgrade (%)	Depth of Treatment (ft)
10-20	2
20-30	3
30-40	4
40-50	5
>50	6

The plasticity index of the expansive soils in this study area ranged from 30 to 60. Therefore, a minimum replacement depth of at least 4 feet appears advisable, if this treatment method is relied upon exclusively.

Backfill material, particularly remolded in-situ soil, should be replaced and compacted with careful moisture and density control. Typical placement conditions should be 92-95 % of the maximum dry density (AASHTO T-99) and at moisture contents between optimum and 5 % above optimum.

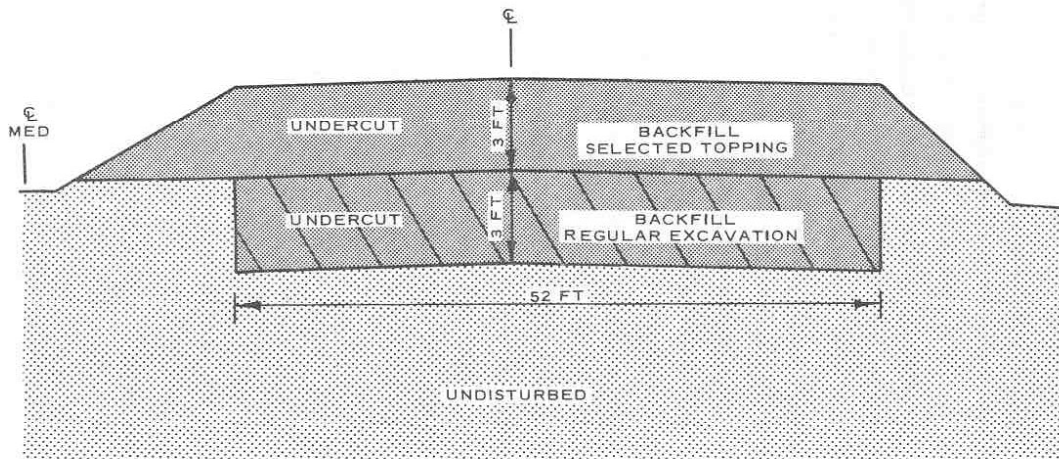


Figure 93: Typical cross-section showing subexcavation and replacement option for an interstate highway application (Snethen, 1979).

A typical example of the use of subexcavation and replacement for an interstate highway is shown in Figure 93. The main-line subexcavation for the top 3 ft extended the full width of the divided four-lane roadway (i.e. shoulder slope to shoulder slope). The backfill material between 3 and 6 feet was confined to the subgrade shoulder lines with the backfill consisting of the remaining subexcavated material placed at a higher moisture content and lower density (Snethen, 1979).

(4) Chemical Alteration

Literally hundreds of chemical compounds have been added to expansive soils in an attempt to alter the clay mineral structure or the clay-water system and thereby prevent volume change. However, due to mixing problems, economics, effectiveness, and practicality, none of these "exotic" compounds are recommended for large-scale routine treatment of swelling soils (Mitchell and Raad, 1974). Lime treatment continues to be the most widely used and effective additive for modification of expansive clays (Snethen, 1979).

While lime treatment may be the most effective and reliable chemical stabilizer for expansive soils, the major obstacle in using this technique is in applying lime to a sufficient depth. Conventional mix-in-place methods can only treat the soil to a depth of 8 to 12 inches. However, lime treatment is well suited for fill construction using potentially expansive soils and to treat expansive soils that are to be used in connection with the subexcavation and replacement approach. For fill applications, the lime can be applied and mixed in the borrow area and for subexcavation and replacement, the lime can be applied in the backfill stockpiles.

In order to consider the use of lime treatment, one must first determine if the soil is lime-reactive and second, how much lime will be necessary to produce the desired volume change reduction. Eads and Grim (1960) have developed a relatively simple testing procedure which helps answer these important questions. The procedure involves mixing the dry soil with varying percentages of lime and adding water. For each lime percentage, the pH is measured

along with the liquid limit and plastic limit. In order for the lime-soil mixture to produce cementitious products (calcium silicate hydrates), the pH must be at or above 12.4. In addition, if the soil is lime reactive, the plastic limit will increase and the liquid limit will decrease as the lime percent is increased so that the plasticity index (difference between liquid limit and plastic limit) or PI drops to less than half of its value in the untreated state. In addition, if the untreated PI is 35 or less, then the treated PI should be 15 or less. Typically, an optimum lime percentage will be determined where the PI is minimized and the pH is also 12.4 or higher.

The Eads and Grim test procedure has been used to evaluate the effectiveness of lime treatment on four soil samples obtained in the expansive soil study zone and the results are presented in Figures 94 through 97. Quick lime for the experiments was obtained from the Cricket Mountain Plant near Delta, Utah operated by Continental Lime, Inc. The pH typically reached 12.4 for lime contents greater than about 3 to 4 %. In addition, the PI, which was between 29 and 65 % for the untreated soil, was typically between 5 and 18 % after the addition of 5 % lime. For higher lime percentages, the PI began to increase or stayed relatively constant. Therefore, a treatment of 5 % lime by weight would appear to be the optimum value. For the four samples tested with 5 % lime the PI was 17, 24, 26 and 27 % of the untreated PI. Since the PI is one of the best indicators of potential swell (Snethen, et al, 1977), these reductions in PI suggest that the lime treatment should be very effective in reducing the potential for swelling in the soils in the study area.

In addition to the pH and Atterberg limit tests using lime treatment, CBR tests were performed on two samples of the subgrade material in the vicinity of the expansive soils. Summaries of the CBR values before and after compaction with 5 % lime are provided in Table 4. Both of these samples were plastic clays and had low CBR values in the untreated state (1 to 4 %); however, in both cases the CBR values increased substantially after treatment (42 to 60.5 %). These tests also demonstrate that the use of lime on these materials leads to a substantial increase in strength and stiffness.

Previous experience has also suggested that lime treatment should not be used if there is more than 5000 ppm soluble sulfate in the soil (Mitchell, 1984). In this case, the sulfate depletes the available lime and can also lead to the formation of two highly expansive minerals, Ettringite and Thaumasite, which can lead to more expansion than that produced by the original soil. To determine if this possibility precluded the use of lime treatment, sodium sulfate tests were performed on 13 disturbed samples collected at regular intervals along the expansive soil study area. The results are summarized in Table 5 and in no case does the sodium sulfate content exceed 140 ppm. These test results indicate that sulfate content should not produce negative performance for the lime-treated soil.

Some references also indicate that gypsum may cause problems similar to those created by sulfates. Therefore, the percent gypsum was also determined for six of the soil samples and the values are tabulated in Table 5. The gypsum content was typically less than 1 %, suggesting that this will not be an issue for this location. Nevertheless, long-term (30 to 60 day) experiments are currently being performed to evaluate the possibility of crystal formation in lime-treated soils.

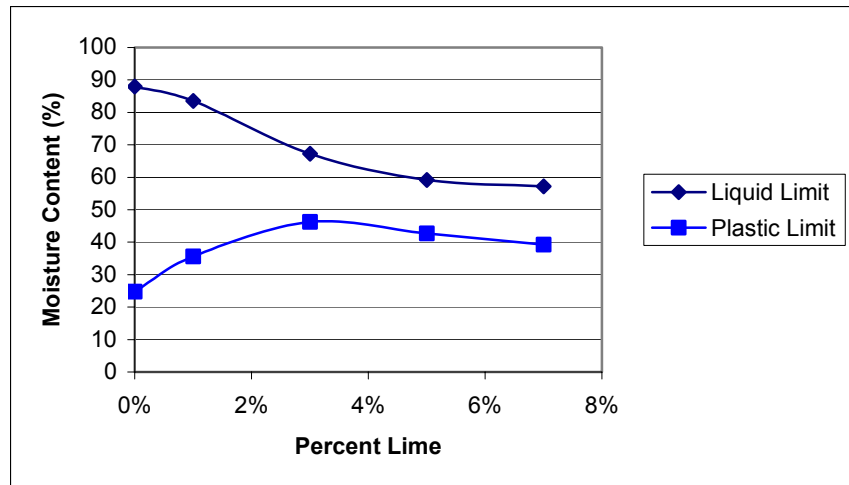
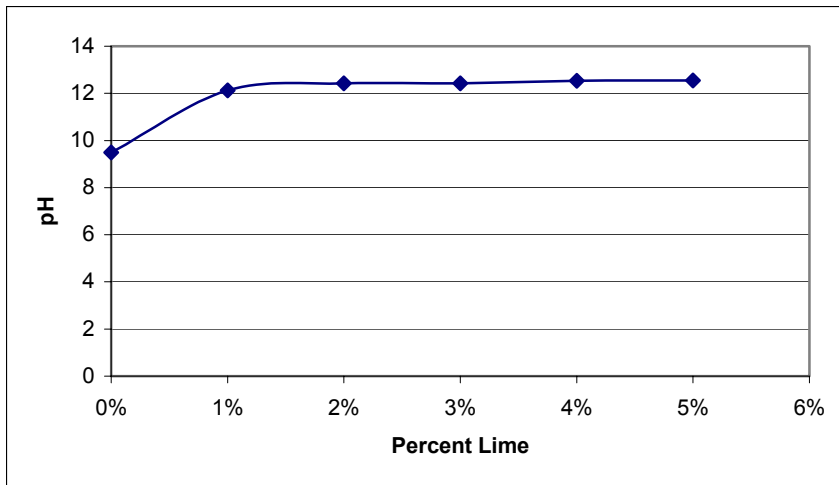


Figure 94: pH and moisture content versus percent lime for sample 4 at 1 ft depth, a yellow clay.

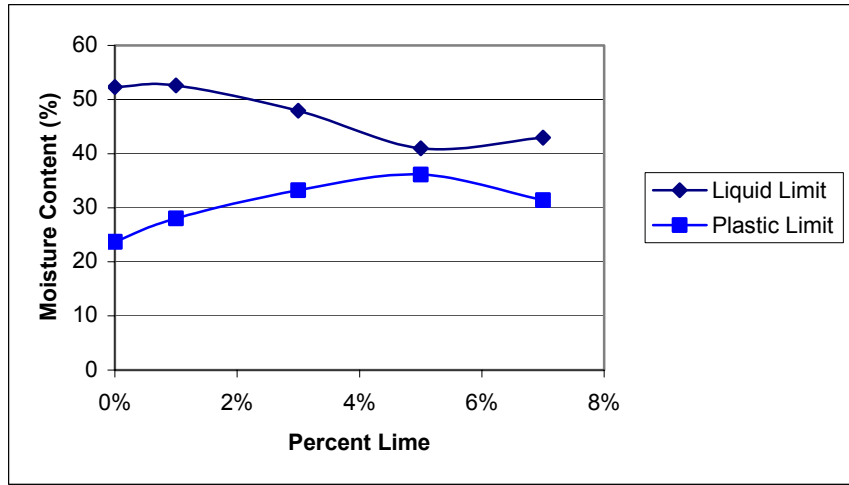
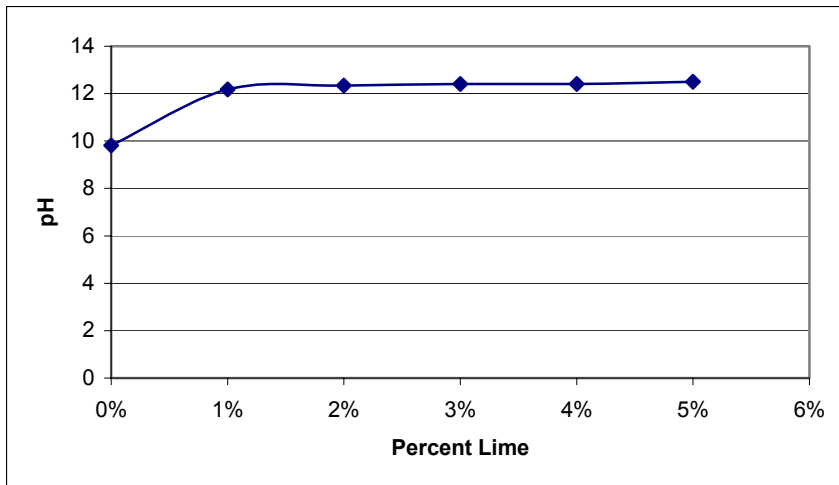


Figure 95: pH and moisture content versus percent lime for sample 2 at 2 ft depth, a brownish gray clay.

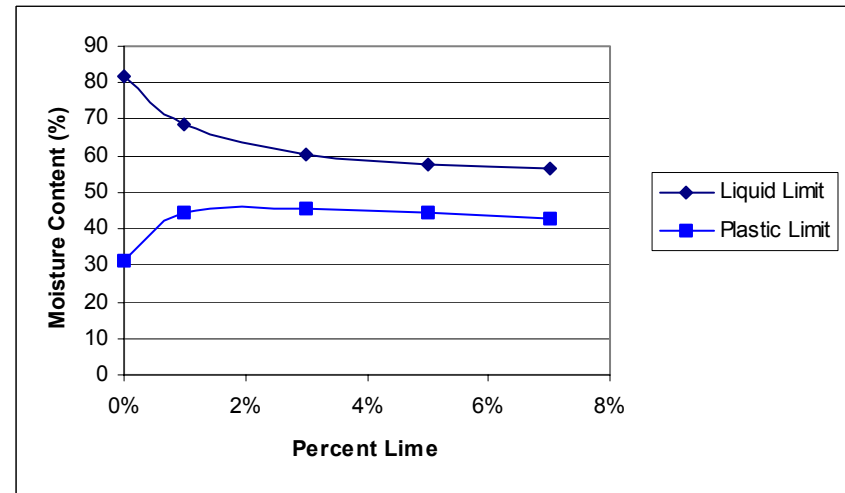
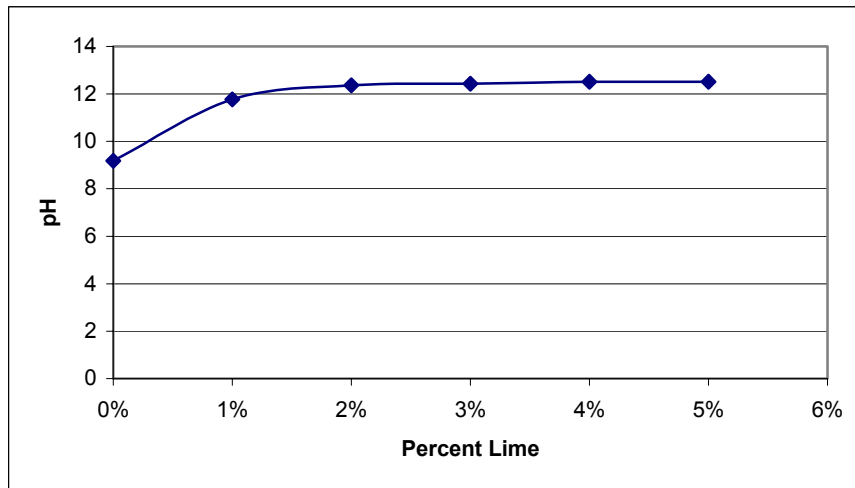


Figure 96: pH and moisture content versus percent lime for sample 6 a 2 ft depth, a green clay.

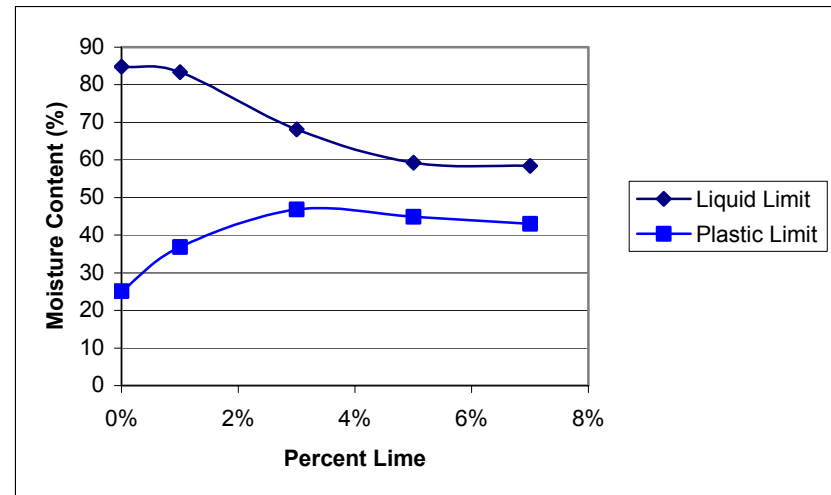
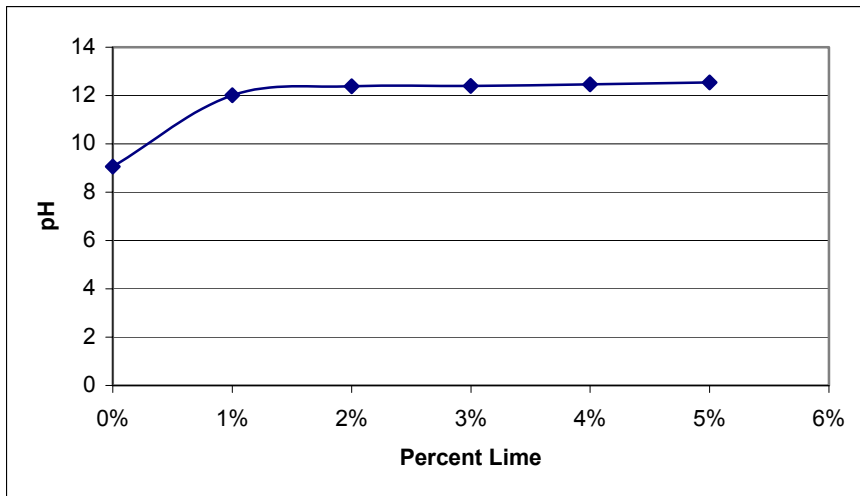


Figure 97: pH and moisture content versus percent lime for sample 13 at 1 ft depth, a green clay.

Table 4: Summary of CBR test results before and after treatment with 5% lime.

Milepost Location	Material Type	Swell On Wetting Before (%)	CBR Value Before (%)	Swell On Wetting Before (%)	CBR Value After (%)
208.820	Yellowish Brown Clay	4.7	1.0	0.4	41.8
209.280	Greenish Gray Clay	1.25	4.0	0.2	60.5

Table 5: Results from Sulfate and Gypsum testing on subgrade samples obtained at approximately 200 ft intervals in the expansive soil zone.

Sample ID	ppm S04	% Gypsum
1	75.99	0.01
2	56.43	-
4	64.50	0.02
5	79.56	0.03
6	55.23	-
7	44.61	0.04
8	82.74	-
9	48.63	-
10	68.58	-
11	63.66	-
12	136.56	-
13	92.55	-
14	50.49	0.03
208.82 CBR	36.50	0.08
209.28 CBR	36.50	0.08

Recommended Treatment Approach for Expansive Soils

In our view the preferred treatment approach for the expansive soil zone would be to provide a combination of subexcavation and replacement acting in concert with horizontal moisture barriers. The existing expansive materials would be excavated to a depth of approximately 3 feet or to intact rock. This material would then be pulverized and mixed with 5 % quicklime by weight. The treated soil would be recompacted in layers at 92 % of the standard Proctor maximum density and at a moisture content between the optimum value and 5 % above optimum. The recompacted zone would extend to the edge of the shoulders as shown in Figure 98. In addition, we recommend that an asphalt moisture barrier be placed over the surface of the backfill and that this barrier extend to a height of 1.5 feet above the expected water level in the drains on either side of the highway and through the median as shown in Figure 98. FHWA

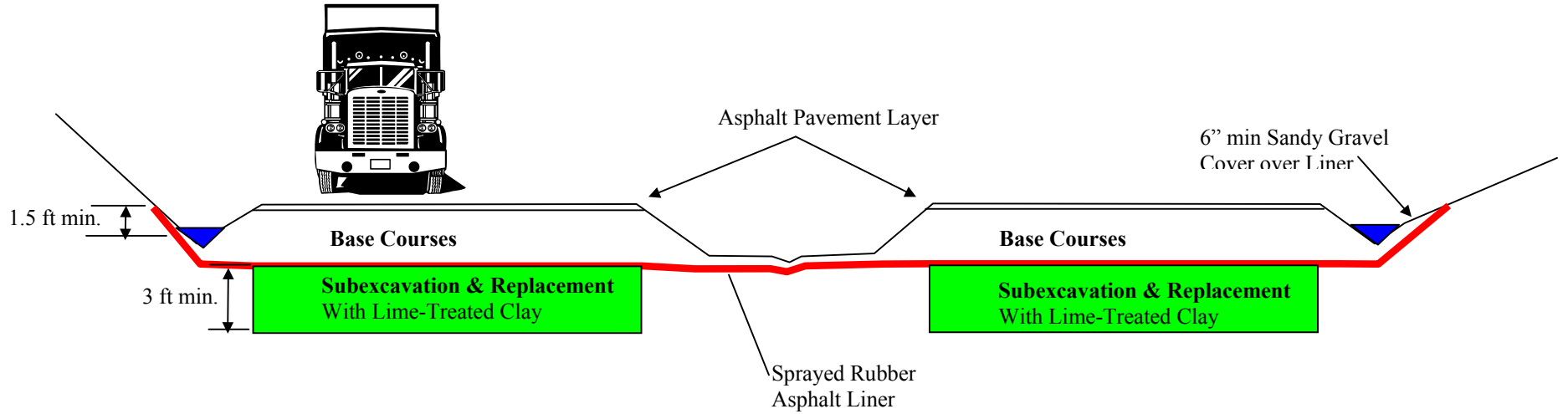


Figure 98: Schematic drawing of typical cross-section in expansive soils treatment zone.

specifications for asphalt rubber liners are provided in the Appendix. Snethen (1979) recommends that a 6 inch thick granular fill layer be placed over the liner to protect it from damage where it would not be covered by the roadway fill as shown in Figure 98. Within this section of the highway drainage ditches are located on either side of the roadway and in the median. The drainage ditches are currently lined with rock or concrete blocks to prevent erosion, however, inspections throughout the course of this study indicate that water frequently ponds in these drain zones and infiltrates into the subsurface materials. This causes the relatively intact tuffs and claystones to weather and expand as the bonds holding the rock together disintegrate.

Increases in curing time and temperature improve the gain in strength for lime-treated soils. Therefore, Nelson and Miller (1992) recommend that the time of construction be scheduled to obtain maximum benefit of summer temperatures before the onset of cold weather. If the soil temperature is less than 60° to 70° F and is not expected to increase for 1 month, then the chemical reactions will be deterred and the benefits of treatment will be minimal (Currin et al., 1976).

Overall Recommendations for the Study Area

Drainage Provisions

The lack of adequate surface drainage is one of the critical factors leading to problems with both collapsible and expansive subgrade soils. Some obvious signs of drainage problems include water ponding in the drainage ditches, soft spots in the ditch, or the presence of plants and weeds that grow best in saturated or submerged environments. These warning signs are present at a number of locations within the study area as noted previously in the section on site investigations. We recommend that these drainage ditches be lined with asphalt with a protective covering of gravel to prevent leakage. In addition, we recommend that cross-drains which pass through the median be designed so that water does not accumulate in the median prior to passing through to the other side of the roadway as was observed at several locations.

Pavement Type and Features

Because of the potential for differential settlement on the roadway, we recommend that asphalt pavement be used in reconstructing the roadway in the study area. Asphalt pavements provide several advantages relative to concrete pavements when expansive and collapsible soils are encountered. First, the pavement provides a "membrane" that helps restrict the infiltration of water into the subgrade. Second, if water does penetrate into the subgrade, the asphalt pavement is more flexible and is better able to accommodate the distortion without significant pavement distress. Third, the remedial repair of a damaged asphalt pavement can be completed quicker and easier than for concrete pavements. However, for highly moisture sensitive soils such as those encountered during this study, the use of asphalt paving alone will be insufficient to prevent pavement distress without the use of moisture barriers and subsurface treatment.

AASHTO guidelines suggest a 10-ft right shoulder and a 4-ft left shoulder. However, the further the infiltration and wetting surface can be maintained from the travel and passing lanes, the

less likelihood of damage to the pavement (Snethen, 1979). Although the 10-ft right shoulder width is sufficient, the 4-ft left should probably be increased to a width of 6 to 8-ft.

Uniformity at Subgrade Discontinuities

Special care should be taken to assure that the subsurface characteristics are more uniform at discontinuities such as cut-fill transitions and around culverts. Within this study, a significant number of bumps and other distress features were located close to these zones. At cut-fill transitions within this study area, significant differences exist in unit weight and compressibility since the cuts tend to be in weathered rock while the fill sections are located on soil.

Minimization of the differences in physical characteristics is the simplest approach to reducing the localized distortions (Snethen, 1979). The subgrade in the transition to the cut section should be ripped or scarified (water added if necessary) and recompact to conditions comparable to the fill sections. A minimum depth of 12 inches should be considered, but preferably, the depth will be between 18 and 24 inches.

Around culvert or in utility or pipeline trenches, the backfill should never consist of coarse-grained material in expansive soil subgrades. Ideally, the backfill material should be a non-expansive cohesive soil compacted to a sufficient degree to minimize moisture infiltration into the trench. If the ideal material is not available, then the natural soil may be used provided it is thoroughly remolded and compacted at a higher moisture content. Consideration may also be given to using lime-stabilized soil.

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APPENDIX

EXAMPLE SPECIFICATIONS FOR ASPHALT RUBBER MEMBRANES FROM ARIZONA DEPARTMENT OF TRANSPORTATION

ITEM - ASPHALT-RUBBER STRESS ABSORBING MEMBRANE (INTERLAYER) (SANI)

The work under this item consists of placing an asphalt rubber stress absorbing interlayer across the full roadway width.

Asphalt Rubber Materials:

The asphalt shall conform to the requirements of Table 705-1 of the Supplemental Specifications for Asphalt Cement AR-1000.

The granulated rubber shall meet the following requirements:

When the mixing procedure involves the intimate contact between the hot asphalt and rubber for a period of five minutes or more, 95 percent of the granulated rubber shall pass The No. 10 mesh sieve and no more than 10 percent shall pass the No. 25 mesh sieve. Where the contact period is less than five minutes, 98 percent of the granulated rubber shall pass the No. 25 sieve. The sieves shall comply with AASHTO Designation M-92.

The specific gravity of the material shall be 1.15 ± 0.02 and shall be free of fabric, wire, or other contaminating materials, except that up to 4 percent of calcium carbonate may be included to prevent the particles from sticking together.

Mixing Asphalt and Rubber-

The material shall be intimately combined as rapidly as possible for such a time and at such a temperature that the consistency of the mix approaches that of a semi fluid material. The temperature of the asphalt shall be between 350 degrees F. and 450 degrees F.

The method and equipment for combining the asphalt and rubber shall be so designed and accessible that the engineer can readily determine the percentage, by weight, of each of time- two materials being incorporated into the mixture.

The proportions of the two materials, by weight, shall be 75 percent \pm 2 percent asphalt and 25 percent \pm 2 percent granulated rubber. After the full reaction described has occurred, the mix shall be cut back with Kerosene. The amount of Kerosene used shall be 5-1/2 percent to 7-1/2 percent, by volume, of the hot asphalt-rubber composition as required for adjusting the viscosity for spraying or better "wetting" of the cover material.

The Kerosene shall have a boiling point Of not less than 350 degrees F. and the temperature of the hot asphalt-rubber shall not exceed 350 degrees F. at the time of adding the Kerosene.

After reaching the proper consistency, application of the material shall proceed immediately and in no case shall the material be held at a temperature over 330 degrees @.- for more than one hour after reaching the proper consistency.

Construction Details:

The existing pavement shall be cleaned in accordance with the requirements of subsection 404-3.01 of the Standard Specifications.

After cleaning and prior to the application of the membrane seal, the existing pavement surface shall be treated with a tack coat.

The hot asphalt-rubber mixture shall be applied at a minimum rate of 0.60 of a gallon per square yard. A rate of 0.75 of a gallon per square yard should be used for estimating purposes (based on 7-1/2 pounds per hot gallon). The distributor should be capable of spreading the asphalt rubber uniformly.

All transverse joints shall be made by placing building paper over the end of the previous application, and the joining application shall start on the building paper. Once the application process has progressed beyond the paper, the paper shall be disposed of as directed by the engineer.

All longitudinal joints shall be lapped approximately 4 inches.

Cover Material (Special):

Immediately after the asphalt-rubber membrane has been placed, Cover Material (Special) should be applied, primarily as a blotter. The rate of application should be only the amount necessary to protect the membrane from construction equipment required for placement of the asphaltic concrete. If traffic is to be carried over the membrane it will be necessary to increase the rate of application to maintain integrity of the asphalt rubber membrane.

For estimating purposes only the rate of application should be 25 pounds per square yard (dry weight). A sample of the cover material shall be submitted for approval at least two weeks- before it is to be used and the engineer will then determine the exact rate of application.

The cover material should be at least as dry as material dried in accordance with the requirements of Section 4.2 of AASHTO T 85 at the time of application.

The cover material (Special) should comply with the following gradation:

Sieve size	Passing
3/8	100
#4	30 - 60
#8	0 - 20
#200	0 - 4

At least 50% by weight of the material retained on the #4 sieve should have at least one rough angular surface produced by crushing.

Rolling:

The cover material shall be rolled with pneumatic tired rollers carrying a minimum of 5,000 pounds on each wheel and a minimum air pressure of 100 pounds per square inch in each tire.

Sufficient rollers shall be furnished to cover the width of the spread with one pass. It is imperative that the first pass be made immediately behind the spreader and if the spreading is stopped for any reason, the spreader shall be moved ahead so that all cover material spread may be immediately rolled. The rolling shall continue until four complete coverages have been made. Final rolling shall be completed within two hours after the application of the cover material.

Removing Loose Cover Material:

The power broom used in removing loose cover material shall be a combination air jet and rotary sweeper type.

Excess loose cover material should be removed prior to placement of the asphaltic concrete. Care should be taken to maintain the broom pressure so that only the loose material will be removed and there will be a minimum dislodgement of imbedded cover material.

Prior to placement of asphaltic concrete a tack coat should be applied if the asphalt-rubber membrane has been subjected to traffic.

Weather Limitations:

Placement of the asphalt rubber stress absorbing membrane (1) shall not be made when the ambient air temperature is less than 50 degrees F,

(2) shall not be placed on other than an absolutely clean pavement, and (3) material shall not be placed if wind conditions are such that a satisfactory membrane is not being achieved.

*U.S. GOVERNMENT PRINTING OFFICE-1979-281-568-17